

DEVELOPMENT OF A STANDARD TEST PROCEDURE FOR USE OF MINERAL
FILLERS TO IMPROVE MARGINAL AGGREGATE

A Thesis

Presented in Partial Fulfillment of the Requirements for the

DEGREE OF MASTER OF SCIENCE

Major in Civil Engineering

in the

UNIVERSITY OF IDAHO GRADUATE SCHOOL

by

WILLIAM AINSWORTH SYLVIES

June, 1967

This thesis of William Ainsworth Sylvies for the Master of Science degree, "Development of a Standard Test Procedure for Use of Mineral Fillers to Improve Marginal Aggregate,"

A. has been reviewed in rough draft form and preparation of the final draft is recommended; permission is granted to proceed with the final examination upon submission of two final draft copies to the Graduate School:

Major Professor _____ Date _____

Committee Members _____ Date _____

_____ Date _____

B. is approved in final draft form:

Head of Department _____ Date _____

Dean of College _____ Date _____

C. has been granted final acceptance after review by the Graduate Council and after successful completion of the final oral examination:

Dean of the
Graduate School _____ Date _____

BIOGRAPHICAL SKETCH OF THE AUTHOR

William Ainsworth Sylvies was born in Los Angeles, California, on October 9, 1925. He attended Roosevelt High School in Honolulu, Hawaii, and graduated in 1943. From November 1943 to October 1945 he served in the U.S. Navy. He entered the California Institute of Technology in March 1946 and received a Bachelor of Science degree in Mechanical Engineering in 1949 and a Bachelor of Science degree in Civil Engineering in 1950. From April 1953 to July 1954 he was employed by the City of Lewiston as Junior Assistant Engineer. He started his present employment with the State of Idaho Department of Highways in August 1954 and received his license as a Registered Professional Engineer in 1957. In September 1963 he was transferred to Moscow as Associate Materials Engineer. He entered the University of Idaho Graduate School in January 1964 and completed the requirements for the degree of Master of Science in Civil Engineering in June 1967, of which this thesis is a part.

ACKNOWLEDGEMENTS

The author wishes to express his appreciation to the State of Idaho Department of Highways for the funds to complete this research project. Appreciation is also extended to the Civil Engineering Department of the University of Idaho for the use of some of their equipment.

Thanks go to Terry R. Howard for his helpful ideas and constructive criticism and for performing the physical testing, to Dick O. Sanchez for his suggestions during development of the procedure, to Philip L. Thomas for his assistance in checking data and drafting the figures, and to Doctor Dale O. Everson for his counsel and assistance with the statistical analysis.

Gratitude is expressed to the following individuals: Professor John J. Peebles, Professor Cecil W. Hathaway and Doctor Donald F. Clifton for their assistance during the investigation and their constructive criticism of the thesis; Mr. Harry L. Day and Mr. Linne F. Erickson for their review and suggestions in the development of the procedure; and to Mr. Robert G. Charboneau for developing the megascopic classification for the aggregate source.

It is a pleasure to acknowledge the help and assistance of Mrs. Sharon Gregg for typing the thesis.

Special thanks are extended to the author's wife, Margaret Eileen Sylvies, whose patience, devotion and understanding made the author's graduate work possible.

SUMMARY

A serious shortage of mineral aggregate suitable for highway construction exists in several major areas of Idaho. Thus there is the urgent requirement to investigate the means of upgrading poor aggregate having high asphalt absorption or stripping characteristics for highway construction needs. As mineral fillers can be used to upgrade asphalt mixtures, determination of the proper mineral filler for such an aggregate will allow substantial savings in highway construction costs.

Four different test methods were evaluated using specially designed well-graded asphalt mixtures. Three different mineral fillers, hydrated lime, portland cement, and limestone dust, were incorporated in varying amounts in constant volume specimens. The specimens were then tested for resistance to deformation, durability, cohesion and strength by the Relative Stability Test, the Moisture Vapor Susceptibility Test, the Minnesota Cold Water Abrasion Test, and the Immersion-Compression Test. These test results were then compared with those from the same tests using control specimens containing no mineral filler. This permitted a standard test method to be developed through statistical analysis for evaluating the effect of mineral fillers in asphalt mixtures containing poor aggregate.

Pertinent conclusions and recommendations are:

1. The standard test method should be the use of the Relative Stability Test and the Immersion-Compression Test for evaluating the improvement in resistance to deformation, resistance to water action, cohesion and strength of asphalt mixtures containing poor aggregate through use of a mineral filler.
2. The 24-hour immersion period at 140° F. in the Immersion-Compression Test should be tentatively established as standard procedure. This will require final substantiation by comparison to the 4-day immersion period at 120° F. during Phase Two of this study.
3. The Moisture Vapor Susceptibility Test should be further analyzed after the procedure has been modified to more nearly duplicate conditions under which asphalt stripping occurs in the field. This will permit a more realistic evaluation of mixture resistance to stripping action.
4. The Minnesota Cold Water Abrasion Test should be dropped from further consideration as it does not provide meaningful results at the higher asphalt contents being used in present-day asphalt surface course construction.

TABLE OF CONTENTS

CHAPTER	PAGE
I. INTRODUCTION	1
Purpose	2
Objective	3
Background	4
Pertinent Literature	4
Mineral Fillers.	5
Test Procedures	9
Hveem Stability Test	9
Moisture Vapor Susceptibility Test	9
Immersion-Compression Test	10
Minnesota Cold Water Abrasion Test	10
Specific Gravity and Air Voids	11
Temperature Control	12
II. BASIC CONSIDERATIONS	14
Terminology	14
Mixture Components	14
Asphalt Cement	14
Mineral Aggregate	15
Megascopic Classification	16
Gradation and Physical Properties	17

CHAPTER	viii PAGE
Mineral Filler	18
Function	20
Types Used	22
Properties	22
Limitations	28
III. METHOD OF INVESTIGATION	29
Use of Constant Total Solid Volume of Asphalt and Mineral Filler.	29
Proportioning Constituents by Volume	29
Theory of Constant Total Volume of Air Voids	30
Method of Adding Mineral Filler	31
Selection of Tests	32
Problem of Asphalt Absorption and Stripping	32
Tests Considered	35
Cohesimeter Test	35
Hveem Stability Test.	35
Moisture Vapor Susceptibility Test.	36
Immersion-Compression Test	36
Minnesota Cold Water Abrasion Test	37
Specific Gravity of Aggregate	37
Types Considered	37
Apparent Specific Gravity	37
Bulk Specific Gravity	38
Bulk-Impregnated Specific Gravity	38
Maximum Specific Gravity (Rice Method)	38

CHAPTER

ix

PAGE

Determination of Air Voids	38
Use of Average Specific Gravity	39
Use of Maximum Specific Gravity (Rice Method)	40
Trial Mixture Series	40
Selection of Amount of Mineral Filler	41
Selection of Trial Asphalt Ratios	42
Repeatability of Values Obtained	43
Effect of Temperature Control	43
Selection of Optimum Asphalt Ratios	44
Test Specimens	46
Use of Different Filler-Asphalt Ratios	48
Asphalt Ratios Used	48
Planned Asphalt Ratios	49
Indicated Asphalt Ratios	50
Relative Stability Test	50
Particle Orientation	51
Constant Total Volume of Specimens	51
Height Measurement	51
Moisture Vapor Susceptibility Test	52
Minnesota Cold Water Abrasion Test	52
Density Requirement	53
Aggregate Gradation	53
Evaluation of Reduced Asphalt Contents	53
Immersion-Compression Test	54

CHAPTER

x
PAGE

Density Requirement	54
Use of 24-Hour Immersion Period	55
IV. TEST RESULTS	56
Relative Stability Test	57
Relative Stability Values	57
Air Voids, Aggregate Weight per Cubic Foot, and Moisture and Volatiles	58
Moisture Vapor Susceptibility Test	58
Relative Stability Values	59
Air Voids, Aggregate Weight per Cubic Foot, and Moisture and Volatiles	60
Minnesota Cold Water Abrasion Test	61
Abrasion Loss	61
Specimens with Planned and Indicated Asphalt Ratios	62
Specimens with Reduced Asphalt Ratios	62
Immersion-Compression Test	64
Unconfined Compression Strengths	64
Specimens with Planned Asphalt Ratios	64
Specimens with Indicated Asphalt Ratios	66
V. ANALYSIS OF RESULTS	68
Trial Mixture Specimens	69
Effect of Temperature Control	69
Optimum Asphalt Ratios	70
Planned Asphalt Ratios	70

CHAPTER	xi PAGE
Relative Stability Test	71
Moisture Vapor Susceptibility Test	73
Minnesota Cold Water Abrasion Test	74
Immersion-Compression Test	75
Comparison of Tests	78
Relative Stability and Moisture Vapor Susceptibility Tests	78
Relative Stability, Moisture Vapor Susceptibility and Minnesota Cold Water Abrasion Tests	81
Relative Stability, Moisture Vapor Susceptibility and Immersion-Compression Tests	81
Minnesota Cold Water Abrasion and Immersion- Compression Tests	83
VI. CONCLUSIONS AND RECOMMENDATIONS	85
Conclusions	85
Recommendations	90
REFERENCES CITED	93
APPENDIX A	99
APPENDIX B	121
APPENDIX C	123
APPENDIX D	136

LIST OF FIGURES

FIGURE		PAGE
1.	Particle Size Distribution of Hydrated Lime	24
2.	Particles Size Distribution of Portland Cement	25
3.	Particles Size Distribution of Limestone Dust	26
4a.	Moderate Longitudinal Cracking Due to Asphalt Stripping	33
4b.	Serious Longitudinal Cracking Due to Asphalt Stripping	33
4c.	"Alligator" Cracking Due to Asphalt Stripping	34
4d.	Pavement Failure Due to Asphalt Stripping	34
5.	Relative Stability Test Stability Value vs Moisture Vapor Susceptibility Test Stability Value	79
6.	Relative Stability Values and Unconfined Compression Strength (Planned Asphalt Ratio) vs Filler-Asphalt Ratio	80
7.	Planned and Indicated Dry vs Immersed Unconfined Compression Strength	82
8.	Relative Stability Values and Unconfined Compression Strength (Indicated Asphalt Ratio) vs Filler-Asphalt Ratio	84
9.	Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing no Mineral Filler	124
10.	Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 1% Hydrated Lime	125
11.	Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 2.5% Hydrated Lime	126

FIGURE		PAGE
12.	Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 3% Hydrated Lime	127
13.	Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 4% Hydrated Lime	128
14.	Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 6% Hydrated Lime	129
15.	Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 2% Portland Cement	130
16.	Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 4% Portland Cement	131
17.	Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 6% Portland Cement	132
18.	Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 2% Limestone Dust	133
19.	Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 5% Limestone Dust	134
20.	Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 8% Limestone Dust	135

LIST OF TABLES

TABLE	PAGE
I. Physical Properties of 85-100 Penetration Asphalt Cement	16
II. Gradation and Physical Properties of Mineral Aggregate from Pit Source Ada 53	19
III. Physical Properties of Mineral Fillers	23
IV. Chemical Analysis for Hydrated Lime and Limestone Dust	27
V. Effect of Temperature Control on the Coefficient of Variation for Stability, Specimen Weight per Cubic Foot, and Percentage of Air Voids of Specimens Containing 2% Limestone Dust	44
VI. Optimum Asphalt Ratio with Respective Values of Relative Stability, Aggregate Weight per Cubic Foot, and Percentage of Air Voids for Different Mixture Combinations Used	47
VII. Planned Asphalt Ratios and Filler-Asphalt Ratios for Different Mixture Combinations Used	49
VIII. Relative Stability Test Values for Relative Stability	58
IX. Relative Stability Test Values for the Average Percentage of Air Voids, Aggregate Weight per Cubic Foot, and Percentage of Moisture and Volatiles in the Specimens	59
X. Moisture Vapor Susceptibility Test Values for Relative Stability	60
XI. Moisture Vapor Susceptibility Test Values for the Average Percentage of Air Voids, Aggregate Weight per Cubic Foot, and Percentage of Moisture and Volatiles in the Specimens	61
XII. Minnesota Cold Water Abrasion Test Values for Abrasion Loss for Specimens having Planned Asphalt Ratios and for Specimens having Indicated Asphalt Ratios	62
XIII. Minnesota Cold Water Abrasion Test Values for Abrasion Loss for Specimens Containing No Filler with Reduced Asphalt Ratios	63

TABLE

PAGE

XIV.	Minnesota Cold Water Abrasion Test Results from Unpublished Idaho Department of Highways Report Entitled "Analysis of Mineral Filler Investigation Pilot Study"	63
XV.	Immersion-Compression Test Values for Average Dry and Immersed Unconfined Compression Strength and Index of Retained Strength for Specimens having Planned Asphalt Ratios	65
XVI.	Immersion-Compression Test Values for Average Dry and Immersed Unconfined Compression Strength and Index of Retained Strength for Specimens having Indicated Asphalt Ratios	67

CHAPTER I

INTRODUCTION

There is now a serious shortage of mineral aggregate suitable for highway base and surface course construction in several major areas of the State of Idaho. This fact is documented by such reports as a materials availability study (1) in Highway Department District Two in south central Idaho which shows that at least four of the nine counties in that district do not have an adequate supply of material suitable for highway base and surface course requirements. Usable aggregate is also scarce in Shoshone County in northern Idaho from Wallace to the Montana State border.

Upon completion of the current Interstate Highway System construction program, these shortages will be even more serious and additional areas will be faced with a deficient supply of usable aggregate. To alleviate this problem, it will be necessary to either haul in good aggregate from sources that may be many miles away, or improve local poor quality aggregate by treatment to a level that is acceptable for use on highway construction projects.

Economic analysis of the two alternatives by the author based upon average construction costs indicates that:

1. a ton of aggregate for surface course material can be hauled approximately 5 miles in lieu of the added cost of treating a ton of plant-mix with 2 per cent of mineral filler,

2. a ton of aggregate for base material can be hauled approximately 25 miles in lieu of the added cost of treating a ton of aggregate with 6 per cent of asphalt and 2 per cent of mineral filler.

Thus, when a source of good mineral aggregate is 5 or more miles farther away from a surface course project (or 25 or more miles farther away from a project requiring base treatment) than is a local source of poor quality aggregate, serious consideration should be given to improving the poor aggregate to acceptable standards by some type of treatment such as the use of a mineral filler.

Mineral fillers can be used to upgrade asphalt mixtures containing poor aggregate as well as to improve asphalt mixtures containing good aggregate. While improving good aggregate mixtures is both desirable and necessary, there is now the more urgent requirement to investigate the means of upgrading poor aggregate having high asphalt absorption or stripping characteristics for highway construction needs. Determination of the proper mineral filler for a source will allow that aggregate to be used on future construction projects and, in many cases, permit substantial savings in highway construction costs.

I. PURPOSE

At present the Idaho Department of Highways does not have a generally accepted procedure for determining the over all improvement in quality of an aggregate treated by a mineral filler. Certain stability, cohesion, and strength tests have been used to evaluate

individual characteristics of a poor aggregate in an asphalt mixture. However, it has not been possible to correlate the results of the different tests to provide a reasonably complete analysis of improvements due to the use of different fillers.

It was thus the purpose of this study to conduct laboratory stability, abrasion, cohesion, and strength tests on various asphalt-mineral filler-aggregate mixtures, using a good aggregate as a standard to reduce the number of variables, and determine if possible which test or tests provide consistently reliable results in appraising the qualities of resistance, durability, and strength in asphalt mixtures. This could permit establishment of a standard test method for evaluating the improvement of poor aggregate sources by use of a mineral filler and provide substantial savings in time and cost in future testing of this type.

II. OBJECTIVE

There were two primary objectives in this investigation. The first objective was the evaluation of four different test methods using specially designed well-graded asphalt mixtures. Three different mineral fillers, hydrated lime, portland cement, and limestone dust, were incorporated in varying amounts in constant volume specimens. The specimens were then tested for resistance to deformation, durability, cohesion, and strength by the Relative Stability Test, the Moisture Vapor Susceptibility Test, the Minnesota Cold Water Abrasion Test, and the Immersion-Compression Test. These test results were then

compared with those from the same tests using control specimens containing no mineral filler. The second objective was the development by statistical analysis of a feasible standard test method for evaluating the effect of mineral fillers in asphalt mixtures containing poor aggregate.

III. BACKGROUND

In the past, other states have improved poor quality aggregate by the addition of a mineral filler to the asphalt-aggregate mixture (2), (3). Results of a preliminary mineral filler investigation recently completed in the University of Idaho Materials Testing Laboratory also indicate that a poor aggregate can be substantially improved by use of a mineral filler (4). However, that study likewise showed that each aggregate source requires individual analysis to determine the type and amount of mineral filler that provides the greatest improvement. The study also pointed out the need for a standard test method. Based upon the recommendations made in the preliminary study, permission was obtained from the Idaho Department of Highways Research Committee to proceed with this investigation.

IV. PERTINENT LITERATURE

Pertinent literature was carefully examined to obtain information relevant to the use of mineral fillers, test methods, specific gravity and air voids, and temperature control in the

development of asphalt mixtures. This knowledge was invaluable in formulating the investigation and analyzing the results.

Mineral Fillers

According to Tunncliffe (5), Clifford Richardson was apparently the first to recognize and describe the importance of mineral filler. Richardson (6) described the function of filler as:

. . . a part of the mineral aggregate of asphalt surfaces for the purpose of rendering the surface more dense, so that it be less acted upon by water, and less liable to interior displacement or movement.

He also recognized that the presence of a mineral filler caused a change in the consistency of the asphalt cement in a mixture.

In 1932 Miller and Traxler (7) reported that the properties of the individual particles of a mineral powder which affect its value as a filler for asphalt mixtures are size, size distribution, shape, and texture. The surface area and void content of the filler are dependent upon those properties of the particles. In 1936 Traxler and Miller (8) also stated that:

The effect of the addition of a mineral powder to a bitumen-- may be most accurately determined by a comparison of the viscosities of the bitumen and the bitumen-filler mixtures. The composition of the mixtures must be considered on a phase-volume basis.

Thus they recommended proportioning the asphalt and mineral filler by volume. Traxler (9) in a separate paper describes a method for measuring the absolute viscosity of an asphalt-mineral filler mixture.

Bollen (10) in 1937 supported the theory that the addition of mineral filler increases the viscosity of asphalt. He also advocated

the use of the "effective amount of mineral filler," the average of the percentages of filler passing the No. 200 sieve as determined by a sieve analysis and by a wash test, in determining the properties of asphalt and mineral filler by weight.

Eleven mineral fillers were evaluated by Warden, Hudson, and Howell (11) in 1959 as to their effect on consistency, ductility, void filling capacity, stability, temperature susceptibility, and resistance to water as a function of the filler-bitumen ratio. The desired quality attributes of a paving filler are enumerated and discussed with respect to their significance in terms of field handling and pavement performance.

Higher asphalt contents can be used in high stability paving mixes containing small amounts of mineral filler according to a paper by Kallas and Krieger (12) in 1960. They also state that small amounts of certain mineral fillers tend to offset the increase in density occurring with increasing compaction temperatures.

In 1961 Kallas and Puzinauskas (13) reported the effects of 11 mineral fillers on density, aggregate voids, Marshall stability, and air voids using constant volume concentrations of the various fillers. They indicated that:

1. the type of mineral filler greatly influences the compaction characteristics of paving mixtures,
2. the filler concentration is equal in importance to type of filler used,
3. pronounced differences in viscosity and shear

susceptibility result when different fillers are used.

Csanyi (14) indicated in 1962 that mineral fillers can perform several functions. One function is that of filling voids in coarser aggregates while another is the creation of a filler-asphalt mastic. He stated that an excess quantity of filler tends to increase stability, brittleness, and proclivity to cracking. Deficiency of filler tends to increase void content, lower stability, and soften the mix.

Kallas, Puzinauskas, and Krieger (15) considered in 1962 the correlation between filler-asphalt mastic viscosity and test properties of specimens that were compacted to nearly constant volume proportions of all mixture components regardless of the filler used.

They concluded in part that:

1. all mineral fillers regardless of type or concentration increase stability or strength properties of compacted asphalt paving mixtures,
2. Marshall stability values reflect both viscosity of asphalt and influence of fillers while Hveem stability values are primarily influenced by the type and concentration of filler.

An excellent review of literature on mineral filler was presented by Tunnicliff (5) in 1962. He proposed a definition for mineral filler, analyzed the different theories regarding the filler-bitumen system, and concluded that "properly proportioned filler is entirely beneficial to any asphalt paving mixture."

Certain conclusions regarding the effects of a mineral filler

on the properties of asphaltic concrete mixes were reached by Csanyi, Cox, and Teagle (16) in 1964. They reported that:

1. the specific effect of a filler on mixes can be determined but that a general statement concerning the effect of fillers on properties such as stability, voids, and cohesion cannot be made,
2. different methods of determining stability differ in their sensitivity to the quantity of filler present in a mix,
3. fillers have a more marked effect on the stability of mixes containing hard aggregate than on those containing softer aggregates.

In 1964 Lowrie (17) reported on the use of hydrated lime to upgrade poor soils and marginal or substandard aggregates in Colorado. He indicated that hydrated lime is used in asphalt paving mixtures primarily to improve the ability of the aggregate to retain a coating of asphalt in the presence of moisture. He also advocated the theory that if the hydrated lime is added to the aggregate, the mixture heated and mixed with the asphalt cement, the principal reaction is between the lime and the asphalt. If, however, the hydrated lime is mixed with the dampened aggregate and allowed to cure for 48 hours before the asphalt is added to the mixture, the principal reaction is between the hydrated lime and the dampened aggregate.

A different concept was advanced by Heukelom (18) in 1965. He stated that the consistency and densification of bituminous mixes are to a first approximation governed by the apparent or bulk volume

of filler and that all test methods which give an indication of the apparent volume are therefore suitable as filler evaluation methods.

Test Procedures

Hveem Stability Test. There is a very high degree of correlation between Hveem stabilometer results and pavement performance according to Hveem and Vallergera (19). This conclusion is supported by McCarty (20) who states that:

. . . the Hveem stability index, because of its close correlation with stability in service, is the most representative characteristic of the mix when laid in the highway.

Griffith and Kallas (21) report that while the Hveem and Marshall stability test methods are empirical, they are being widely used and have been extensively correlated with asphalt pavement performance. Minor (22) also remarks that case histories of several selected bituminous pavements show that their service characteristics, particularly as related to plastic deformation, were accurately forecast by the stabilometer and cohesiometer.

Moisture Vapor Susceptibility Test. Skog and Zube (23) report that a test was developed in the late 30's to aid in the design of paving mixtures which would be resistant to the effects of moisture vapor. After correlation with actual pavement performance and some modification, the present Moisture Vapor Susceptibility Test evolved. According to Kari (24), moisture vapor can cause a failure in the cohesion bond and at the aggregate-asphalt interface resulting in stripping and migration of the asphalt. The Moisture Vapor Susceptibility Test indicates the extent to which the stabilometer and

cohesiometer values of bituminous mixtures are affected by moisture in the vapor state entering the mixture from a wet subgrade or other sources.

Immersion-Compression Test. The 4-day, 120° F. immersion period in the standard Immersion-Compression Test can generally be relied upon to distinguish between satisfactory and unsatisfactory fillers according to Carpenter (25). However, Goldbeck (26) questions whether the test truly indicates a probable failure of pavement in service and recommends that a longer immersion period be used. Goode (27), in contrast, states that "the immersion-compression method of designing bituminous paving mixtures has proved completely satisfactory." Further support of the test is given by Lowrie (17) who states that after 17 years of laboratory experience and field correlation with the Immersion-Compression Test, the Colorado Department of Highways has acquired considerable confidence in its reliability.

Minnesota Cold Water Abrasion Test. Swanberg and Hindermann (28) conclude that the cold water abrasion test can be used satisfactorily to evaluate the durability of asphalt-aggregate mixtures with respect to the stripping action of water. They state that there appears to be a satisfactory correlation of the test with field performance and recommend a maximum permissible abrasion loss of 15 per cent.

Specific Gravity and Air Voids

In 1947 Kampf and Raisch (29) advanced the idea that high voids cause rapid hardening of the asphaltic cement. They also concluded that both the filler (material passing the No. 200 mesh sieve) and the asphaltic cement go directly into the voids of the mixture, and that the voids rather than the surface area of the aggregate determine the asphaltic cement and filler requirements.

Ricketts and others (30) state that because aggregates absorb bitumen to a variable extent, no single one of the conventional specific gravities has proved satisfactory for general use with porous aggregates in bituminous mixtures. An erroneous aggregate specific gravity can cause a bituminous mix to be too lean or too rich, with attendant unsatisfactory characteristics. They describe a bulk-impregnated specific gravity which is a function of the ratio of bitumen to water absorption of an aggregate and report that it has been found satisfactory for general use with porous aggregates.

Weight-volume relations are widely used as criteria for the design of bituminous paving mixtures according to Benson and Subbaraju (31). The determination of specific gravity of the individual constituents of the paving mixture is therefore a necessary part of the design procedure. They report that the bulk-impregnated specific gravity procedure and the aerosol and vacuum procedure have both been found to yield values of specific gravity considered to closely represent the probable weight-volume relationships in bituminous mixtures.

McLeod (32) points out that the magnitude of the errors in values reported for "per cent voids in the mineral aggregate," "per cent air voids," and "per cent voids filled with bitumen" for compacted paving mixtures made with highly absorptive aggregate can be quite serious. He recommends that the three values be evaluated on the basis of the ASTM bulk specific gravity of the aggregate making full allowance for the asphalt absorbed into the aggregate. In a second paper, (33) he makes the same recommendation and presents reasons for rejecting both the ASTM apparent and the effective specific gravity of the aggregate as a basis for pavement design.

The void properties of compacted dense-graded bituminous paving mixtures are at least as important as their stability insofar as pavement performance is concerned according to McLeod (34) in a third publication. He also concludes that the most significant void requirements for compacted mixtures are air voids and voids in the mineral aggregate.

Temperature Control

Kiefer (35) shows that changes in the compaction temperature of bituminous concrete mixtures produce changes in the specific gravity, per cent voids, stabilometer value, and cohesiometer value of the compacted mix. He indicates that the compaction temperature used in the compaction and testing of bituminous concrete mixtures must be carefully controlled in order to give reproducible results.

Variations in the mixing and compacting viscosities of asphaltic concrete produced changes in Marshal stability, flow value,

specific gravity, and voids of the compacted mixtures according to Bahri and Rader (36). They also report that some of the variations were large enough to require selection and control of proper mixing and compacting viscosities.

CHAPTER II

BASIC CONSIDERATIONS

I. TERMINOLOGY

Terminology in the highway construction field is fairly well specialized with most terms having a specific meaning. However, several terms need further explanation to clarify their meaning.

The terms "mix," "mixture," and "asphalt mixture," designate asphalt-aggregate mixtures or asphalt-mineral filler-aggregate mixtures developed to simulate plant-mix surface course. The terms "aggregate," "asphalt," and "filler," are used to denote mineral aggregate, asphalt cement, and mineral filler, respectively.

II. MIXTURE COMPONENTS

Asphalt cement, mineral aggregate, and mineral filler (when used) are the essential components of a mixture. Each component is discussed in appropriate detail in the following sections.

Asphalt Cement

American Society for Testing and Materials (ASTM) Designation D-8-63 describes asphalt as:

A dark brown to black cementitious material, solid or semisolid in consistency, in which the predominating constituents are bitumens which occur in nature as such or are obtained as residua in refining petroleum.

It also defines asphalt cement as:

A fluxed or unfluxed asphalt specially prepared as to quality and consistency for direct use in the manufacture of bituminous pavements, and having a penetration at 25 C. (77° F.) of between 5 and 300, under a load of 100 gm. applied for 5 seconds.

Asphalt cement is a strong, readily adhesive, cementing agent which imparts controlled flexibility to mixtures of mineral aggregate or mineral aggregate and mineral filler with which it is combined. Such mixtures are plastic in nature, reasonably waterproof, and quite durable (37).

An 85-100 penetration asphalt cement was used for all tests in this investigation. This grade and type of asphalt is representative of the asphalt used with mineral filler treatment on actual construction projects in the field.

The test results listed in Table I on page 16 indicate the physical properties of the 85-100 penetration asphalt cement used in this study. The results are the average of four separate complete tests on 40 gallons of asphalt. The individual results of the four complete tests are contained in Appendix B on page 122.

Mineral Aggregate

Idaho Department of Highways Pit Source Ada 53 is located 2 miles west of Milepost 63.6 on the Gowen Field Road near Boise. This is one of the best sources of mineral aggregate in southwestern Idaho and hence the aggregate was selected for use in this investigation. Aggregate from this source has been used on a number of different highway construction projects which have since had good performance records.

TABLE I
PHYSICAL PROPERTIES OF 85-100 PENETRATION ASPHALT CEMENT

PROPERTY	RESULT
TESTS ON ORIGINAL ASPHALT	
Penetration of Orig. Sample at 77° F., 100 gm., 5 sec.	89
Flash Point, P.M.C.C. (°F.)	500+
Kinematic Viscosity at 275° F. (cs)	267
Specific Gravity at 77°/77° F.	1.019
Solubility in CCl ₄ (%)	99.74
Spot Test, Heptane Xylene Equivalent at 35% Xylene	Negative
TESTS ON RESIDUE FROM THIN FILM LOSS ON HEATING	
Loss on Heating at 325° F., 5 Hours (%)	0.0
Penetration at 77° F., 100 gm., 5 sec.	56
Ratio of Thin Film L.O.H. Pen./Orig. Pen. (%)	62.9
Ductility at 77° F., 5 cm/min (cm)	100+

Megascopic Classification. The megascopic classification for the source was developed by R. G. Charboneau, District Geologist for Highway Department District Three in southwestern Idaho. The analysis was done at the request of the author.

According to Charboneau, Ada 53 is from the Gowen Terrace series of the Caldwell-Nampa sediments of the Pleistocene to Recent epoch of the Quaternary period. These are late terrace gravels that are primarily medium-grained to coarse-grained aggregates principally granitic in origin. The plus No. 4 aggregate is estimated to be:

1.	Granite and Biotite Granite	-	36.2%
2.	Porphyry (Different types)	-	15.5%
3.	Granodiorite	-	14.8%
4.	Olivine Basalt	-	6.2%
5.	Basalt	-	5.6%
6.	Diorite	-	5.5%
7.	Andesite	-	3.7%
8.	Quartz Porphyry	-	3.6%
9.	Other	-	8.9%

while the minus No. 4 aggregate is estimated to be:

1.	Quartz	-	45%
2.	Feldspar	Orthoclase - 40%	
		Plagioclase - 60%	40%
3.	Mica	-	5%
4.	Other	-	10%

Gradation and Physical Properties. The aggregate, which was obtained from a stockpile, had been crushed to meet the Idaho Department of Highways 1965 Standard Specifications for both a Class "D" Plant-mix Surface Course and a 3/4-in. Maximum Type "B" Aggregate Base Course. This gradation is representative of that which would be used with mineral filler treatment in both asphalt treated base courses and plant-mix surfacing courses on construction projects in the field. Since the aggregate obtained from the stockpile was evenly graded, it was used as received without further processing.

The physical properties of the aggregate were determined by standard American Association of State Highway Officials (AASHO) test procedures or by standard Idaho Department of Highways (Idaho T Method) test methods. The specific test methods are listed in Section I-B of Appendix A on pages 101 and 102. A complete discussion of the Idaho Degradation Test is contained in an article by H. L. Day (38).

The gradation and physical properties of the mineral aggregate are listed in Table II on page 19. The fine gradation is the average of four wash tests made on different parts of the entire sample.

Mineral Filler

There is no generally accepted definition of mineral filler. Csanyi (14) has defined mineral filler as that fraction of a mineral aggregate, flour, or dust present in an asphalt mixture that passes the No. 200 sieve. However, in another report (16), filler is defined as the fraction of the mix material passing the No. 200 sieve, either as part of the aggregate or of the so-called mineral filler. Tunncliffe (5) takes a different approach and defines mineral filler as mineral material which is suspended in asphalt cement resulting in a cement of stiffer consistency. AASHO Designation M-17-42 states that mineral filler shall consist of limestone dust, portland cement, or other inert mineral matter and shall meet certain gradation requirements including at least 65 per cent passing the No. 200 sieve. In this report mineral filler is used to denote either hydrated lime, portland cement, or limestone dust.

TABLE II

GRADATION AND PHYSICAL PROPERTIES
OF MINERAL AGGREGATE FROM PIT SOURCE ADA 53

GRADATION		PHYSICAL PROPERTIES	
<u>Sieve Size</u>	<u>Per Cent Passing</u>	<u>Property</u>	<u>Test Result</u>
3/4"	100	Liquid Limit (%)	No Value
5/8"	100	Plastic Limit (%)	Non Plastic
1/2"	98	Plastic Index (%)	Non Plastic
3/8"	87	Sand Equivalent (%)	58
No. 4	60	Fine Specific Gravity	2.60
No. 6	50	Coarse Specific Gravity	2.57
No. 8	42	Average Specific Gravity	2.59
No. 20	25	Coarse Aggregate Water Absorption (%)	1.1
No. 30	21	Asphalt Absorption by Aggregate(%)	1.34
No. 40	16	Los Angeles Abrasion Test (% Wear)	23.8
No. 50	11	Idaho Degradation Test	
No. 100	6	Original % Minus No. 200	3
No. 200	4.0	Final % Minus No. 200	10
Dust Ratio (%) 25		Original Sand Equivalent (%)	58
		Final Sand Equivalent (%)	33
$\frac{\% \text{ No. 200}}{\% \text{ No. 40}} \times 100 = \text{Dust Ratio}$			

Function. Tunnickliff (5) indicates that there are two general theories regarding the function of mineral filler. The first theory proposes that the filler, having a relatively very fine particle size distribution, serves primarily to fill the voids between the coarser aggregate particles thereby reducing the size of the voids and increasing the density and stability of the compacted mixture. Thus the void space in the coarser aggregate is filled with mineral particles passing the No. 200 sieve making the size of the voids smaller and the density of the mass greater. The second theory suggests that filler is mineral material which is in colloidal suspension in the asphalt cement resulting in a cement with a stiffer consistency. Thus the asphalt cement is filled with colloidal mineral matter which increases its viscosity. A complete discussion of both theories is contained in the articles and papers reviewed in Chapter I under "Pertinent Literature" on pages 4 to 13.

There is no general agreement as to which of the two theories is the most valid. It is quite possible that both theories are appropriate depending on:

1. the method by which the mineral filler is added to the asphalt mixture,
2. the type, amount, and particle size of the filler,
3. the amount of asphalt present in the mixture.

In support of the first theory Csanyi (14) states:

In conventional mixing operations, observations and tests have shown that only a small portion of the filler particles are individually coated while a larger portion tends to form small agglomeration of particles that are coated, and the remainder

adhering to the surfaces of the aggregate particles are coated as integral parts of the aggregates. Regardless of the manner in which filler particles are coated, they do, however, serve to fill void spaces between aggregate particles.

However, he also reports that:

It is generally recognized that filler particles may be readily suspended in asphalts and that the finer particles will remain in suspension longer than coarser particles. . . . When a filler becomes suspended in an asphalt, a mastic is created that has a lower consistency, as measured by the penetrometer, than the original asphalt cement. . . . A mastic of this type is harder, stiffer, tougher, and possesses a lower temperature susceptibility than the original asphalt cement.

Since the portion of filler particles completely coated in conventional mixing operations would be suspended in asphalt, the above statements would also tend to support the second theory.

Kallas and Puzinauskas (13) support both theories with the statement:

For example, if filler is sufficiently fine and the mixture contains sufficient amount of asphalt, the mineral fines will be located within the asphalt films and therefore change the binder properties. In other words, it will act as a filler within the asphalt itself, since it will replace a certain amount of asphalt in the mixture. On the other hand, if the aggregate in the mixture is well-graded, or if the mixture contains relatively small quantities of asphalt and the asphalt films are relatively thin, at least a part of the filler will contribute in producing the contact points between the aggregate particles. This dual role is peculiar to the mineral filler in the asphalt paving mixture and distinctly separates it from other mineral components of that mixture.

There is, however, reasonably general agreement that the addition of properly proportioned mineral filler to the asphalt mixture provides increased stability and better durability. According to Tunnicliff (5) who supports the second theory, a particle of filler will absorb a layer of asphalt which will entirely enclose the particle.

Because of the surface energy of attraction between the particle surface and the asphalt, the adsorbed layer of asphalt assumes a different, stiffer consistency. The increased stability is believed due to the stiffened asphalt while the improved durability is attributed to the nature of the surface energy of attraction of the adsorbed film of asphalt for the particle surface. The existing surface energy must be supplanted by energy from other sources such as water, dust, grit, thermal expansion or thermal contraction before surface course deterioration can take place.

Types Used. Hydrated lime was selected because of the past favorable experience with its use in Colorado as reported by Lowrie (17) and in Utah and Wyoming as indicated by Eager (2). The favorable increase in Hveem Stability values shown by hydrated lime for increasing ratios of filler to asphalt as contained in the paper by Kallas, Puzinauskas, and Krieger (15) also supported its use. Portland cement and limestone dust were chosen because of their ready availability for Idaho highway construction projects and their suitability as mineral filler according to Warden, Hudson, and Howell (11). The lack of sensitivity to water action of all three fillers as reported by Kallas and Puzinauskas (13) further supported the decision to use them in this study. Time and money limited the investigation to the use of three mineral fillers.

Properties. Hydrated lime is calcium hydroxide in the form of a fine powder with about 95 per cent passing the No. 200 sieve.

Limestone dust is primarily calcium carbonate ground to a fine powder with about 85 per cent passing the No. 200 sieve. Portland cement in this investigation is Type I portland cement.

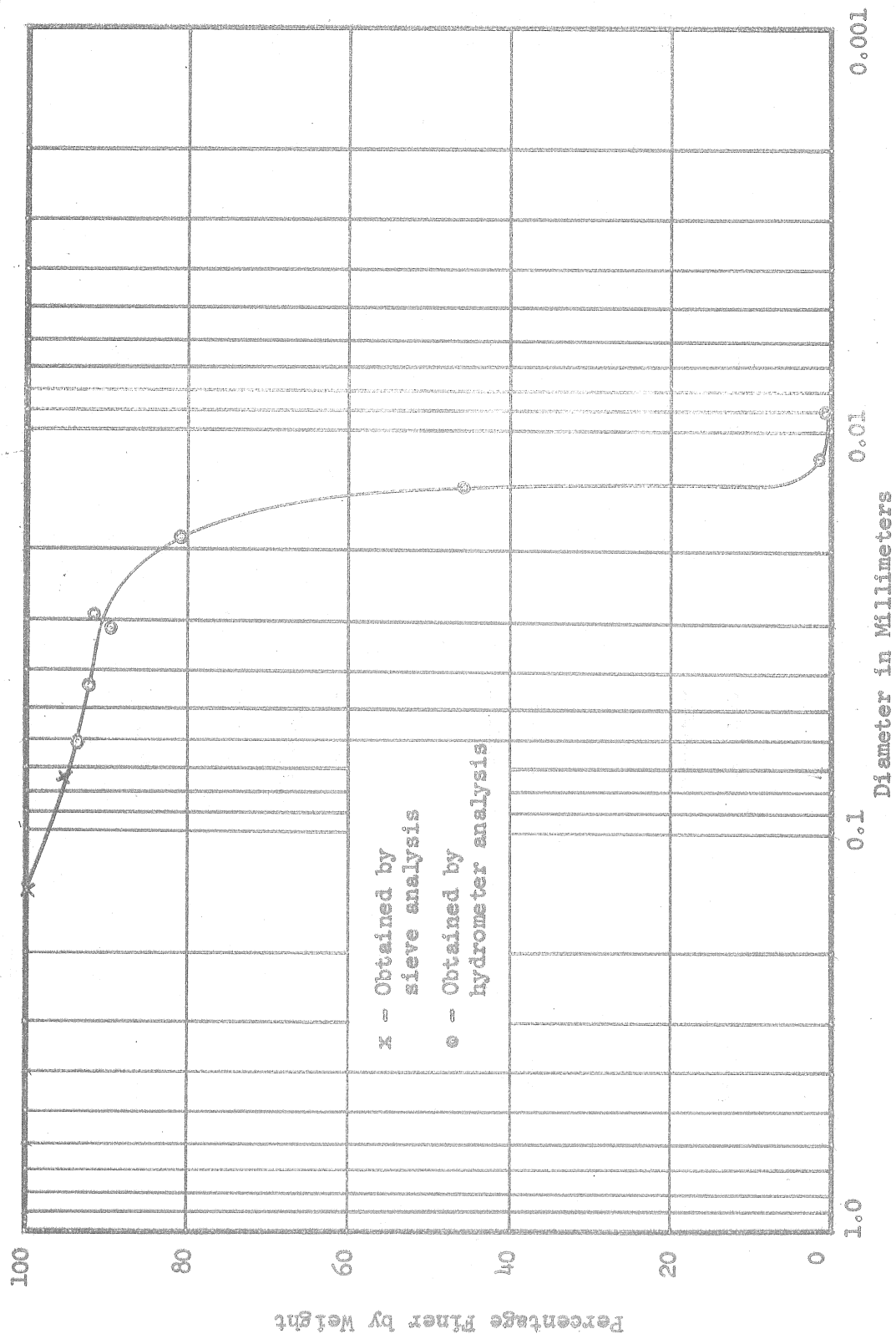
The bulk density, hygroscopic moisture content, specific gravity, and particle size distribution were determined for each of the three mineral fillers as used in this study. These values are shown in Table III and Figures 1, 2, and 3, respectively on pages 24 to 26.

TABLE III
PHYSICAL PROPERTIES OF MINERAL FILLERS

Filler	Bulk Density gm/cm ³	Hygroscopic Moisture Content (%)	Specific Gravity
Hydrated Lime	0.75	1.01	2.45
Portland Cement	1.47	1.01	3.08
Limestone Dust	1.53	0.0	2.70

Bulk densities in grams per cubic centimeter were measured by agitating a 100 cubic centimeter graduate filled with loose dry filler until no further consolidation could be observed. This indicates the bulking property of the dry filler according to Kallas and Puzinauskas (13).

The hygroscopic moisture content was measured in conformance with AASHTO Designation T-88-57. The specific gravity of the limestone dust was obtained in accordance with AASHTO Designation T-100-60 while the specific gravity of the hydrated lime and of the portland cement was



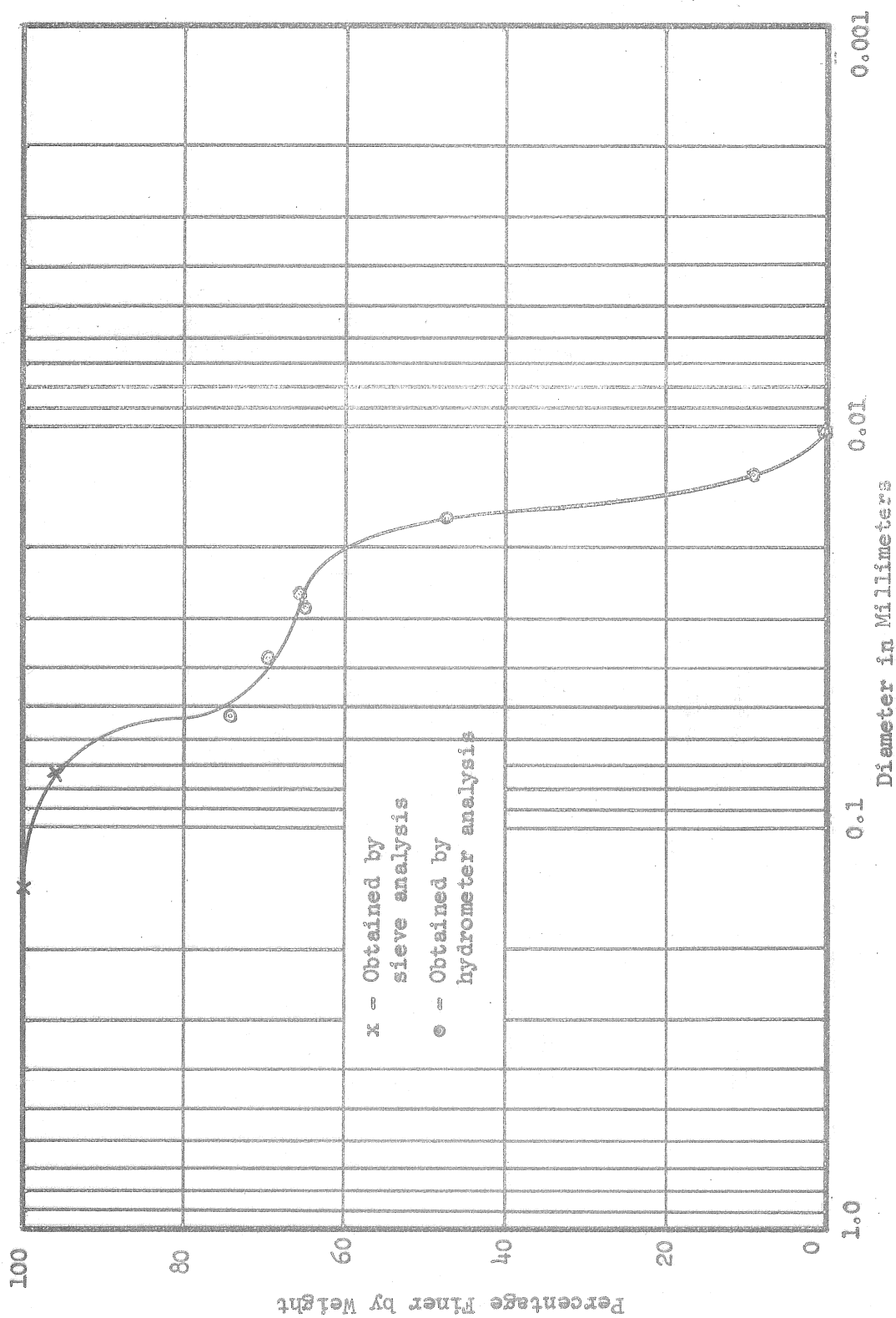


Figure 2. Particle Size Distribution of Portland Cement

determined according to AASHTO Designation T-133-45. Particle size distribution was obtained by the method of grain size analysis described by Lambe (39).

A chemical analysis was submitted for the hydrated lime and the limestone dust by their respective manufacturers. These are shown in Table IV. Such an analysis was not available for the portland cement. However, the cement was supplied from a pre-tested lot that met specifications listed under AASHTO Designation M-85-60.

TABLE IV
CHEMICAL ANALYSIS FOR HYDRATED LIME AND LIMESTONE DUST

	Hydrated Lime (%)	Limestone Dust (%)
Free moist.	0.2	--
Insol. HCL	2.51	--
R_2O_3	0.45	--
Fe_2O_3	0.118	0.14
Al_2O_3	0.332	--
$CaCO_3$	1.43	97.62
$Ca(OH)_2$	92.94	--
SO_4	Trace	--
MgO	1.90	--
CO_2	0.63	--
IGN. Loss	23.45	--
Avail. CaO	68.02	--
$MgCO_3$	--	0.70
Insoluble & Silica		1.54

Limitations. Warden, Hudson, and Howell (11) point out that the filler in the asphalt mixture must be non-critical; that is, variations in the filler content in normal hot-plant mixing operations must not cause undesirable qualities in the physical properties of the surface course. Furthermore, the quantity of filler used must not create operational problems in the mixing, placing and compaction of the surface course. They also indicate that the filler must be non-hygroscopic and not lump, cake, or bridge in the supply bins.

Csanyi (14) points out that as filler content increases, the brittleness and tendency of the surface course to dry out and crack also increases. Thus it is quite possible that poor quality aggregate which now tends to dry out and crack could be further aggravated in that direction when combined with a mineral filler.

Use of a mineral filler may also be limited by economic considerations. Addition of a high percentage of filler may increase the asphalt requirement of the mixture. This, together with the cost of the filler, causes an increase in cost per ton of mixture that could preclude the use of this type of treatment.

CHAPTER III

METHOD OF INVESTIGATION

The investigation was designed to establish which existing routine laboratory tests are the most significant in determining accurate values for resistance to deformation, resistance to water action, durability, cohesion, and strength of asphalt mixtures. Thus no effort was made to analyze the chemistry involved between asphalt, mineral filler, and mineral aggregate. That knowledge would be most valuable but it is beyond the present means of the Idaho Department of Highways to make the information available for the great majority of construction projects. Detailed petrographic analyses are made on certain mineral aggregate sources, but that information is also limited to special problems. Hence this study did not include such analyses.

I. USE OF CONSTANT TOTAL SOLID VOLUME OF ASPHALT AND MINERAL FILLER

Using a constant total solid volume of asphalt and mineral filler with a constant solid volume of aggregate in constant total volume specimens should permit accurate control of air voids. Such a procedure has the advantage of minimizing the effects of variation in air void content on the different test results.

Proportioning Constituents by Volume

Traxler and Miller (8) have stated that:

. . . it has become clear that a consideration of the weight relationships of the liquid and solid phases cannot lead to a clarification and understanding of the stabilizing action of the filler. A bituminous mixture must be considered as composed of a certain percentage by volume of bitumen combined with a certain percentage by volume of the mineral or solid.

That recommendation is supported by McLeod (32) who points out that:

Because it provides a set of simple, easily understood basic criteria not only for the design of paving mixtures but also for analyzing their performance in the field, it is believed that the proper approach to the design of dense graded bituminous concrete is not on a weight basis but of volume.

Kallas and Puzinauskas (13) also agree with this idea as they remark that since different fillers vary widely in specific gravities or densities, it is believed that the use of the volumetric basis provides for better comparison between specimens containing different fillers.

Theory of Constant Total Volume of Air Voids

It is generally accepted among those associated with highway construction that a high air void content in an asphalt pavement mixture can promote hardening of the asphalt and subsequent cracking of the pavement. It was therefore decided to design the asphalt mixtures with a constant total volume of air voids so as to minimize the effects due to variation in air void content. This could best be accomplished by using specimens of constant total volume containing a constant solid volume of mineral aggregate and a constant total solid volume of asphalt and mineral filler. Varying the individual solid volumes of asphalt and filler to achieve the same constant total solid volume of those two constituents would thus permit a clear and more direct

evaluation of the effects of different fillers and filler concentrations.

The total volume of air voids will remain constant in specimens of constant total volume with constant solid volumes of aggregate, asphalt, and filler. However, varying the individual solid volumes of asphalt and mineral filler will vary the number, but not the total volume, of air voids in the mixture (15). Hence, while the size and shape of the individual air voids will change and the surface area of aggregate and filler particles exposed to the air voids will vary, the total volume of air voids in the mixture should not change. Thus the effects due to variation in air void content would still be minimized.

Method of Adding Mineral Filler

In making an asphalt mixture, the mineral filler can be:

1. mixed with the asphalt first and the combination added to the mineral aggregate,
2. mixed with the mineral aggregate first before the asphalt is added.

In conventional hot-plant mixing operations the aggregate is first dried and then placed in the heated mixing chamber, the mineral filler added and mixed with the aggregate, and then the asphalt cement added and mixed with the aggregate and filler to complete the operation. Accordingly, in this investigation the filler was mixed with the aggregate first before the asphalt was added in order to simulate field conditions.

II. SELECTION OF TESTS

In selecting the physical tests to be used, only existing routine laboratory tests were considered. The different tests were carefully reviewed to determine which ones best indicate the resistance of an asphalt mixture to failure from asphalt absorption or stripping.

Problem of Asphalt Absorption and Stripping

Absorption is the filling of part or all of the volume of the water permeable pore space within the individual aggregate particles with asphalt thereby reducing the amount of asphalt available for the cementing function (32). This is caused partly by the moisture in the aggregate pores being converted to expanding gas when heated in the mixing process and drawing asphalt into the aggregate pores when the gas is converted back to moisture during the cooling process. Stripping is defined as the loss of asphalt films from aggregate surfaces in the presence of moisture (40).

Stripping especially can cause serious longitudinal or "alligator" cracking in a surface course which can lead to eventual failure. Figures 4a, 4b, 4c, and 4d on pages 33 and 34 illustrate the progression of this type of cracking due to stripping action. Sections of the surface course involved were removed from the highway and inspected thoroughly to determine the cause of the problem. Examination of the asphalt mixture by the author and other engineers proved that the cracking was primarily the result of asphalt stripping.

In view of the primary concept of this study, no effort was made to evaluate the individual problems of asphalt absorption or



Figure 4a. Moderate Longitudinal Cracking Due to Asphalt Stripping



Figure 4b. Serious Longitudinal Cracking Due to Asphalt Stripping



Figure 4c. "Alligator" Cracking Due to Asphalt Stripping



Figure 4d. Pavement Failure Due to Asphalt Stripping

stripping as such. The many complex reactions involved in these problems are beyond the scope of this investigation.

Tests Considered

Five different tests were evaluated as to suitability for use. Four were selected for the reasons given in the following discussions.

Cohesimeter Test. The Cohesimeter Test measures the cohesion of compacted asphalt mixtures by measuring the force required to break or bend the sample as a cantilever beam. The test produced somewhat erratic results in the preliminary study (4), and tests made by the California Highway Department over a 20-year period show that there is no dependable direct or consistent correlation between test procedures such as the cohesimeter and the performance of surface courses. Therefore, any test results that have good correlation with the cohesimeter cannot be expected to correlate with surface course performance (19). In view of this, the Cohesimeter Test was dropped from further consideration.

Hveem Relative Stability Test. The Hveem Relative Stability Test is a triaxial test that determines the resistance to deformation of compacted asphalt mixtures by measuring the lateral pressure developed from applying a vertical axial load. The specimens are not exposed to moisture vapor during the test.

The Idaho Department of Highways has successfully used the Hveem Relative Stability Test for surface course design since 1950 to predict the ability of the surface course to withstand permanent deformation under traffic loads. Since Hveem stability values are primarily

influenced by the type and concentration of filler (15), and in view of the high degree of correlation between the test results and surface course performance (19) (20) (21), the Hveem Relative Stability Test was selected for use. The test will often be referred to in this study as the Relative Stability Test or just the stability test.

Moisture Vapor Susceptibility Test. The Moisture Vapor Susceptibility Test consists of the Hveem Relative Stability Test being performed on specimens which have been exposed to moisture vapor for a stated period of time. The Moisture Vapor Susceptibility Test has been used successfully by the California Highway Department for a number of years to predict the effect of moisture vapor on the stability of an asphalt mixture. Because it has been successfully correlated with actual pavement performance (23), this test was selected for use.

Immersion-Compression Test. The Immersion-Compression Test measures the loss of cohesion resulting from the action of water on compacted asphalt mixtures. It compares the unconfined compressive strength of freshly molded and cured specimens with the unconfined compressive strength of duplicate specimens that have been immersed in water under prescribed conditions.

The Idaho Department of Highways has been using the Immersion-Compression Test for several years to determine the effect of water on the compressive strength of asphalt mixtures. While there has not as yet been adequate opportunity to correlate test results with actual surface course performance, it is believed that the test provides reliable results. As other states have expressed confidence in this

test (2) (17), it also was selected for use.

Minnesota Cold Water Abrasion Test. The Minnesota Cold Water Abrasion Test utilizes the abrasive action of one specimen upon another due to rotation of the specimens in a water-filled Deval cylinder. The test was selected primarily because it is an abrasion type of test and because it has been used successfully by the Minnesota Highway Department to evaluate the durability of mixtures to asphalt stripping (28).

III. SPECIFIC GRAVITY OF AGGREGATE

Specific gravity is normally considered to be the ratio of the weight of a substance in air to the weight in air of an equal volume of water. However, because mineral aggregate is a porous media which absorbs asphalt in part of its pores, several types of specific gravity can be used to determine the density of aggregate in an asphalt mixture.

Types Considered

Four different types of specific gravity are commonly used to determine the density of mineral aggregate. Each type will provide a different value for the aggregate density.

Apparent Specific Gravity. Apparent specific gravity is the ratio of the weight in air of a given volume of the impermeable portion of a permeable aggregate to the weight in air of an equal volume of water (30). Use of this specific gravity in connection with an asphalt mixture assumes that all water permeable aggregate pores are filled with asphalt. This is an erroneous assumption.

Bulk Specific Gravity. Bulk specific gravity is the ratio of the weight in air of a given volume of a permeable aggregate to the weight in air of an equal volume of water (30). Use of this specific gravity in connection with an asphalt mixture assumes that no water permeable aggregate pores are filled with asphalt. This also is an erroneous assumption.

Bulk-Impregnated Specific Gravity. Bulk-impregnated specific gravity is the ratio of the weight in air of a given volume of a permeable aggregate to the weight in air of an equal volume of water minus the weight of the volume of asphalt absorbed by pores which are permeable to it (30). Use of this specific gravity in connection with an asphalt mixture assumes that the asphalt absorption can be accurately measured. This is quite difficult to do.

Maximum Specific Gravity (Rice Method). The maximum specific gravity is the ratio of the weight in air of a given volume of asphalt-coated aggregate and mineral filler (if present) to the weight in air of an equal volume of water (41). Use of this specific gravity in connection with an asphalt mixture permits the determination of the actual specific gravity of the mixture particles as they comprise the specimen.

IV. DETERMINATION OF AIR VOIDS

The accurate measurement of air voids in an asphalt mixture is a necessary part of the design of a durable surface course. As the measured value of the air voids depends on the specific gravity of the

mixture, it is most important that the type of specific gravity used determine the true specific gravity of the mixture particles.

Use of Average Specific Gravity

Average specific gravity is the sum of the bulk specific gravity multiplied by the percentage of plus No. 4 mineral aggregate and the apparent specific gravity multiplied by the percentage of minus No. 4 mineral aggregate in the specimen. Average specific gravity when multiplied by the unit weight of water provides the unit solid weight of aggregate from which the percentage of voids in the mineral aggregate is calculated. By subtracting the volume of asphalt present in the mixture from the percentage of voids in the mineral aggregate, the percentage of air voids in the mixture is determined. This has been the standard method used by the Idaho Department of Highways in the past to determine the percentage of air voids in an asphalt mixture.

Use of average specific gravity assumes varying amounts of asphalt absorption by the aggregate since it is based upon the coarse aggregate absorbing no asphalt and the fine aggregate absorbing all possible asphalt. Recent experience by the Idaho Department of Highways has shown that use of the average specific gravity gives a lower percentage of air voids than is actually the case. Accordingly, the Idaho Department of Highways no longer uses this method in the design of plant-mix surface courses.

Use of Maximum Specific Gravity (Rice Method)

Maximum specific gravity when multiplied by the unit weight of water provides the unit solid weight of aggregate plus asphalt plus mineral filler. By dividing the unit solid weight of the constituents into the unit weight of the specimen and subtracting the dividend from 100 per cent, the actual percentage of air voids in the asphalt mixture can be determined. This procedure is now the standard method used by the Idaho Department of Highways for determining the percentage of air voids in an asphalt cement mixture.

Use of the maximum specific gravity method does not depend upon measurement of the asphalt absorbed by the aggregate particles. It determines the specific gravity of the asphalt coated particles as they exist in the mixture and permits an accurate calculation to be made of the actual air voids in the specimen. Accordingly, the maximum specific gravity method was selected for use in this study.

V. TRIAL MIXTURE SERIES

In designing an asphalt mixture, an estimate of the amount of asphalt required is made by use of the Centrifuge Kerosene Equivalent (42). A set of trial mixture specimens is then developed using various asphalt ratios greater than and less than the Centrifuge Kerosene Equivalent value. Relative stability value, aggregate weight per cubic foot, and percentage of air voids are ascertained for each trial mixture specimen and the different respective values are plotted on a graph. The optimum asphalt ratio is then determined for the mixture

from the best combination of values for relative stability, aggregate weight per cubic foot, and percentage of air voids. This process is called a trial mixture series or simply a trial mix.

A set of trial mixture specimens was developed for each type and amount of mineral filler investigated. From the results as explained above, it was possible to determine the optimum asphalt ratio and corresponding relative stability value, weight per cubic foot, and percentage of air voids for each particular type of mixture. These values were then used for comparison at a later time with corresponding values from actual test specimens having asphalt ratios specifically designed to be above or below optimum.

Selection of Amount of Mineral Filler

From past experience it was known that the practical limit on amounts of hydrated lime, portland cement, and limestone dust that can be used in an asphalt mixture is approximately 3 per cent for hydrated lime, 4 per cent for portland cement, and 5 per cent for limestone dust. Use of greater amounts of filler either cause the mixture to be too dry within the range of practical asphalt ratios or do not normally improve the desired physical properties of the mixture.

By using greater amounts of each type of filler, however, asphalt mixtures could be developed that should have higher relative stability values, higher abrasion losses, and lower compressive strengths than the corresponding values for specimens containing the practical limit of filler.

Moreover, by using smaller amounts of filler than the practical limit, asphalt mixtures could be developed that should have lower relative stability values, higher abrasion losses, and lower compressive strengths than the corresponding values for specimens containing the practical limit of filler. Hence the use of three different amounts of each type of mineral filler would provide a greater range of test values from which the different tests could be more effectively evaluated.

Trial mixture specimens were made for 1%, 3%, and 6% hydrated lime by weight of aggregate and the optimum asphalt ratios were determined in accordance with Section II of Appendix A on page 102. However, upon making a test specimen for each combination, it was obvious that the use of 6 per cent hydrated lime made that particular specimen too dry. Specimens containing 4 per cent and 5 per cent hydrated lime were then made and it became obvious that 4 per cent hydrated lime was the maximum amount that could be used. Thus the types and amounts of mineral filler by weight of aggregate used to make the test specimens for the four different tests were:

1. No filler
2. 1%, 2.5%, and 4% hydrated lime
3. 2%, 4%, and 6% portland cement
4. 2%, 5%, and 8% limestone dust

Selection of Trial Asphalt Ratios

The Centrifuge Kerosene Equivalent value was determined for each of the above mixture combinations. From those values it was evident

that trial asphalt ratios of 4%, 5%, 6%, and 7% were appropriate for all mixture combinations except that for 4 per cent hydrated lime which required trial asphalt ratios of 5%, 6%, 7%, and 7.3%.

Repeatability of Values Obtained

Considerable difficulty was experienced in achieving repeatability of values for relative stability, specimen weight per cubic foot, and percentage of air voids in four of the trial mixture specimen series. Those series were performed as many as four times before representative values could be attained. It was thus decided to examine the effect of temperature control on these values at critical stages of specimen development.

Effect of Temperature Control

A set of five specimens containing 2 per cent limestone dust was prepared on each of 2 different days with no temperature control. Next, a set of ten specimens was prepared on the same day with accurate temperature control and a set of five specimens was prepared on each of 2 different days with accurate temperature control. All specimens were prepared and tested for relative stability, specimen weight per cubic foot, and percentage of air voids. This permitted evaluation of both temperature control and preparation of specimens on different days as compared with the same day.

The coefficient of variation for the different conditions and properties is shown in Table V on page 44. The results of this limited investigation do not seem to indicate that temperature control

improves the repeatability of the values involved. However, at least two reports (35) (36) state that variations in the mixing and compacting viscosities of asphalt concrete produce changes in stability, density, and voids of the compacted mixtures. Thus the mixing and compacting temperatures of all specimens were controlled throughout this research project.

TABLE V

EFFECT OF TEMPERATURE CONTROL ON THE COEFFICIENT OF VARIATION
FOR STABILITY, SPECIMEN WEIGHT PER CUBIC FOOT, AND
PERCENTAGE OF AIR VOIDS IN SPECIMENS
CONTAINING 2% LIMESTONE DUST

No Temperature Control	Coefficient of Variation For Specimen		
	Stability	Wt./Ft. ³	Percentage of Air Voids
Set I of Five Specimens	6.76	0.29	3.14
Set II of Five Specimens	3.92	0.41	11.78
Temperature Control			
Set of Ten Specimens	2.75	0.30	7.10
Set I of Five Specimens	7.34	0.55	6.91
Set II of Five Specimens	5.13	1.07	14.59

Selection of Optimum Asphalt Ratios

Optimum asphalt ratio is defined as the percentage of asphalt in a mixture calculated by weight of aggregate plus mineral filler (if present) that provides the most advantageous combination of relative stability, aggregate weight per cubic foot, and percentage of air voids

values for the mixture. The optimum asphalt ratio is also called the optimum asphalt content. However, since optimum asphalt ratio is the more commonly used term, it shall be used in this study.

The minimum permissible value for relative stability is 35 for heavy traffic while the percentage of air voids should be between 3 and 5 per cent (43). The aggregate weight per cubic foot is chosen to be as high as possible within the limitations designated for the relative stability and percentage of air voids. The average values of relative stability, aggregate weight per cubic foot, and percentage of air voids plotted against asphalt ratio for each different type and amount of filler used are shown in Appendix C on pages 124 to 135.

Several factors had to be considered in selecting optimum asphalt ratios for this investigation. The optimum asphalt ratios for the control mixtures containing no filler, 2.5 per cent hydrated lime, 4 per cent portland cement, and 5 per cent limestone dust were selected first, keeping in mind:

1. the problem of repeatability for stability, aggregate weight per cubic foot, and air voids in test specimens to be developed,
2. the specification limitations on those values,
3. the rate of change of the slope of the relative stability curve,
4. the variations in filler-asphalt ratio planned for the other six mixture combinations.

Analysis of the different curves in Appendix C indicated that the percentage of air voids for the control mixtures would have to range between 5 and 8 per cent and that the minimum permissible relative stability value would have to be lowered to 30 for light traffic for some combinations of mixtures. Using a minimum relative stability value of 30 would not create any problems, however, since the important factor would be the relationship of other stability values to the control stability value rather than the magnitude of the control stability value itself.

Optimum asphalt ratios for the remaining six mixture combinations were chosen only to obtain the most favorable combination of relative stability, aggregate weight per cubic foot, and percentage of air voids for the particular mixture. These were needed for comparison with the respective values obtained by planned variations in filler-asphalt ratio for the same mixture combinations. The optimum asphalt ratios for all ten mixture combinations together with their respective relative stability, aggregate weight per cubic foot, and air voids values at the optimum asphalt ratio are shown in Table VI on page 47.

VI. TEST SPECIMENS

Test specimens for the four different tests were prepared using the types and amounts of mineral filler by weight of aggregate indicated on page 42. The test specimens were developed after all trial mixture series had been completed and the corresponding optimum asphalt ratios selected.

TABLE VI

OPTIMUM ASPHALT RATIO WITH RESPECTIVE VALUES OF RELATIVE STABILITY,
 AGGREGATE WEIGHT PER CUBIC FOOT, AND PERCENTAGE OF
 AIR VOIDS FOR DIFFERENT MIXTURE
 COMBINATIONS USED

Filler	Optimum Asphalt Ratio (%)	Relative Stability	Aggregate Wt./Ft. ³ (Lb./Ft. ³)	Air Voids (%)
No filler	7.0	41	128.9	7.2
1% hydrated lime	7.0	37	129.5	6.1
2.5% hydrated lime	6.8	37	130.0	7.3
3% hydrated lime	6.8	30	130.0	6.1
4% hydrated lime	6.7	30	131.0	7.2
6% hydrated lime	7.0	32	127.0	8.2
2% portland cement	7.0	46	130.4	5.7
4% portland cement	6.8	33	131.4	5.8
6% portland cement	6.1	37	134.3	6.2
2% limestone dust	6.9	37	130.5	6.4
5% limestone dust	6.3	30	132.8	5.2
8% limestone dust	5.9	31	133.4	6.1

Mixture combinations containing no filler, 2.5 per cent hydrated lime, 4 per cent portland cement, and 5 per cent limestone dust were used as basic control mixtures. Optimum asphalt ratios were also used in the basic control mixtures. A constant total solid volume of asphalt and mineral filler was determined for each control mixture containing a filler, and that same total solid volume of asphalt and filler was used in the other two mixture combinations containing that

filler. By replacing a given solid volume of asphalt with an equal solid volume of filler (or vice versa) to achieve the planned percentages of each type of mineral filler, it was possible to examine the effect of three different filler-asphalt ratios on the different test values for mixtures containing a particular type of filler.

Use of Different Filler-Asphalt Ratios

The filler-asphalt ratio is defined in this investigation as the ratio of the volume of mineral filler used in the mixture to the volume of asphalt used in the mixture. Experience has shown that the ratio of these two constituents fluctuates to a moderate degree (and sometimes fairly widely) during normal hot-plant mixing operations. Thus, it is necessary to determine the effect of variation in design ratio of the two constituents on the physical properties of the surface course. Furthermore, by designing planned variations in this ratio, it would be possible to see which test or tests best evaluate the difference in values. Analysis of these two major factors is of paramount importance to the success of this study.

Asphalt Ratios Used

The principle of using the highest possible asphalt content in a paving mixture consistent with maintaining adequate stability has been proven sound by experience (12). It has been shown, for instance, that the higher the asphalt content, the longer the fatigue life of the surface course (44). Thus the asphalt ratios used were chosen to be as high as possible consistent with other requirements.

An example of the method used to determine the asphalt ratio in the test specimens for each different filler combination is contained in Appendix A on page 107. The asphalt ratios and filler-asphalt ratios for each different mixture combination are contained in Table VII .

TABLE VII

PLANNED ASPHALT RATIOS AND FILLER-ASPHALT RATIOS FOR
DIFFERENT MIXTURE COMBINATIONS USED

Filler	Asphalt Ratio (%)	Filler- Asphalt Ratio
No filler	7.0	
1% hydrated lime	7.5	0.054
2.5% hydrated lime	6.8	0.146
4% hydrated lime	6.1	0.257
2% portland cement	7.5	0.084
4% portland cement	6.8	0.184
6% portland cement	6.1	0.302
2% limestone dust	7.6	0.095
5% limestone dust	6.3	0.281
8% limestone dust	5.1	0.538

Planned Asphalt Ratios. Asphalt ratios shown in Table VII are referred to as planned asphalt ratios since they were calculated as indicated above. All test specimens for the Relative Stability Test and the Moisture Vapor Susceptibility Test were developed using only planned asphalt ratios.

Indicated Asphalt Ratios. By accident both the Minnesota Cold Water Abrasion Test and the Immersion-Compression Test were initially completed with all specimens having optimum asphalt ratios of 7.0%, 6.8%, 6.8%, or 6.3% for specimens containing no filler, hydrated lime, portland cement, or limestone dust, respectively. Results from those two particular sets of tests will be referred to as for specimens with indicated asphalt ratios.

Both the Minnesota Cold Water Abrasion Test and the Immersion-Compression Test were run a second time using the planned asphalt ratios shown in Table VII. Results from those two particular sets of tests will be referred to as for specimens with planned asphalt ratios.

VII. RELATIVE STABILITY TEST

Relative stability is defined as the ability of an asphalt surface course to resist plastic deformation under repeated stress conditions developed by vehicle traffic or intermittent loads (45). Thus the Hveem stabilometer furnishes a method of measuring the internal friction of an asphalt mixture under load. Stability test values are relative in that they indicate lateral deformation relative to axial load.

The Relative Stability Test procedure is outlined in detail in Section IV of Appendix A on page 108 . Five test specimens were made for each set since the use of more test specimens does not further decrease by any significant amount the probability of a larger value of Student's "t" at the 95 per cent confidence level (46).

Particle Orientation

Compaction of the test specimen by means of the kneading compactor permits simulation of the actual particle orientation obtained when the mixture is compacted both initially on a construction project and later by the action of traffic. When a surface course is constructed in the field, the rolling process by the breakdown, intermediate, and finishing rollers tends to orient the mixture particles in a particular pattern depending upon the particle type, size, and shape. The orientation is continued by the compactive effort of traffic. Experience has shown that the kneading compactor develops the same general pattern of particle orientation in the mixture while compacting it as a test specimen in the laboratory (47).

Constant Total Volume of Specimens

As the kneading compactor supplies a constant pressure during compaction of the specimen, it was planned to vary the leveling load after compaction to obtain the necessary constant total volume for all test specimens containing a particular type of filler. This procedure was not entirely successful in all cases in that mixtures containing 4 per cent hydrated lime and 8 per cent limestone dust could only be compressed to a minimum volume that was slightly larger than that of the rest of the specimens. However, the amount of difference was not believed serious enough to warrant modifying the procedure.

Height Measurement

Considerable difficulty was experienced in obtaining an accurate measurement of the height of specimen needed for constant volume.

Since the application of the leveling load usually formed the top surface in a plane that was not exactly parallel to the plane of the bottom surface, it was necessary to take the average of heights measured at the edge of the quarter points as the height of specimen.

VIII. MOISTURE VAPOR SUSCEPTIBILITY TEST

The Moisture Vapor Susceptibility Test is the Relative Stability Test performed on specimens that have been subjected to a continuous 75-hour period of exposure to moisture vapor. As in the Relative Stability Test, the specimens for the Moisture Vapor Susceptibility Test are tested for relative stability and percentage of moisture and volatiles in the specimen. The detailed test procedure is contained in Section V of Appendix A on page 112. Comments on number of test specimens, particle orientation, constant total volume of specimens, and height measurement made previously regarding the Relative Stability Test are applicable to the Moisture Vapor Susceptibility Test.

IX. MINNESOTA COLD WATER ABRASION TEST

The Minnesota Cold Water Abrasion Test measures the loss of material from the test specimens due to the abrasive action of the specimens upon each other. Loss of material is termed abrasion loss and is measured as a percentage of the original specimen weight. The complete test procedure is contained in Section VI of Appendix A on page 114. Each set tested was made up of eight specimens 2 in. in diameter and 2 in. in height.

Density Requirement

The test procedure requires the density of each specimen to be nine-tenths of the density of the corresponding Relative Stability Test specimen with the same type and amount of mineral filler. This necessitated accurate density determination for the Relative Stability Test specimens.

Aggregate Gradation

The asphalt mixture was screened over a heated 1/2-in. sieve after all specimen components were thoroughly mixed to separate out the plus 1/2-in. aggregate particles. Thus only that part of the mixture passing the 1/2-in. sieve was used to mold the specimens.

Evaluation of Reduced Asphalt Contents

Review of both initial sets of Minnesota Cold Water Abrasion Test results revealed that all of the test values were concentrated in the low range of from 1.1 per cent to 7.0 per cent abrasion loss with all but one value being in the range of from 1.1 per cent to 3.7 per cent abrasion loss. This was consistent with the results of previous tests performed in the University of Idaho Materials Testing Laboratory (4).

Study of the original paper (28) from which the test was developed showed that the asphalt ratios used range from 3.75 per cent to 5.3 per cent with the great majority being from 4.0 per cent to 5.0 per cent. As the asphalt ratios in this investigation range from 5.1 per cent to 7.6 per cent, the question arose as to whether this test

was meaningful when higher asphalt ratios were used.

A short series of tests were thus conducted using specimens containing no mineral filler with asphalt ratios of 6.0%, 5.0%, 4.0%, and 3.0% to determine roughly at what critical asphalt ratio the abrasion loss exceeded 15 per cent, the criteria established by the Minnesota Highway Department for maximum permissible abrasion loss. Once the critical asphalt ratio was roughly determined, another series of tests was conducted using asphalt ratios of 0.2 per cent difference on either side of the critical ratio to examine that range more closely.

X. IMMERSION-COMPRESSION TEST

The Immersion-Compression Test compares the unconfined compressive strength of dry specimens with the unconfined compressive strength of duplicate specimens that have been immersed in water. Immersed specimens are also commonly referred to as wet specimens.

A compression test on unconfined specimens is primarily a measure of cohesion rather than of internal friction (48). The test procedure is outlined in detail in Section VII of Appendix A on page 118. Five test specimens were used for reasons explained in connection with the Relative Stability Test on page 50.

Density Requirement

Density of the test specimen is controlled entirely by the test procedure. Test specimens are cylinders 4.0 in. in diameter and 4.0 ± 0.1 in. in height which are compressed by the double plunger method using a load of 3000 psi for 2 minutes. Thus enough mixture

must be placed in the mold to form a specimen meeting the above conditions.

Use of 24-Hour Immersion Period

Wet specimens were immersed in water for 24 hours at 140° F. before being tested for compressive strength rather than being immersed for 4 days at 120° F. Both options are contained in the standard test procedure. The 24-hour immersion period was selected because others have shown that there is very close agreement between the 1-day values at 140° F. and the 4-day values at 120° F. (48). It is believed that the 1-day immersion period at 140° F. represents a condition that is more severe than those normally encountered in the field (13).

CHAPTER IV

TEST RESULTS

All pertinent results for the four different tests are listed in the various tables contained in this chapter. The tables are placed at the end of the section involved.

It is to be remembered that both the Relative Stability Test and the Moisture Vapor Susceptibility Test provide relative stability values and percentages of moisture and volatiles in the specimens. However, the specimens in the Moisture Vapor Susceptibility Test have been exposed to moisture vapor while the specimens in the Relative Stability Test have not been so exposed.

There are two different sets of test results for the Minnesota Cold Water Abrasion Test and also for the Immersion-Compression Test. For each of the two different tests, one set of results will be for specimens having planned asphalt ratios while the other set of results will be for specimens having indicated asphalt ratios. Use of the two different asphalt ratios has been explained on pages 49 and 50.

Coefficient of variation has been used to show the variability of individual values in sets of relative stability values and also in sets of unconfined compression strength for both wet and dry Immersion-Compression Test specimens. Coefficient of variation is a quantity used to evaluate variability in results from separate tests of the same type. It is defined as the standard deviation of the individual values expressed as a percentage of the mean of the individual

values. Expressed as a formula it would be:

$$\text{Coefficient of variation} = \frac{100 \times \text{standard deviation}}{\text{mean value}}$$

To know whether or not a particular coefficient of variation is too large requires experience with similar data. It is a relative measure of variation (46).

I. RELATIVE STABILITY TEST

Relative stability values and their corresponding coefficient of variation are the two most important results from the Relative Stability Test. Percentage of air voids, aggregate weight per cubic foot, and percentage of moisture and volatiles in the specimens are supporting data. All test results are used for comparison with corresponding values from the Moisture Vapor Susceptibility Test and the trial mixture series.

Relative Stability Values

Individual stability values for the different mixture combinations are shown in Table VIII on page 58 together with the average stability values and the coefficients of variation for the individual values. Average stability values for three of the four control mixtures (no filler, 4 per cent portland cement and 5 per cent limestone dust) are reasonably close to the design values shown in Table VI on page 47 for those combinations. However, the average stability value for the remaining control mixture (2.5 per cent hydrated lime) is much lower than its counterpart in Table VI.

TABLE VIII
RELATIVE STABILITY TEST VAULES FOR RELATIVE STABILITY

Filler	Relative Stability Values	Average Relative Stability Value	Coefficient of Variation (%)
No filler	45, 45, 45, 40, 51	45	8.6
1% hydrated lime	20, 34, 20, 31, 30	27	24.3
2.5% hydrated lime	32, 24, 28, 23, 25	26	13.8
4% hydrated lime	43, 42, 40, 40, 40	41	3.4
2% portland cement	21, 33, 21, 22, 25	24	20.8
4% portland cement	30, 30, 28, 36, 25	30	13.5
6% portland cement	37, 43, 45, 42, 43	42	7.1
2% limestone dust	15, 20, 18, 19, 18	18	10.4
5% limestone dust	24, 25, 28, 22, 32	26	14.9
8% limestone dust	42, 41, 44, 46, 45	44	4.7

Air Voids, Aggregate Weight per Cubic Foot, and Moisture and Volatiles

Average values for percentage of air voids, aggregate weight per cubic foot, and percentage of moisture and volatiles in the specimens are included in Table IX on page 59. Average values for aggregate weight per cubic foot and percentage of air voids for the four control mixtures are reasonably close to the design values in Table VI on page 47 for those combinations.

II. MOISTURE VAPOR SUSCEPTIBILITY TEST

Relative stability values, their corresponding coefficient of variation, and the percentage of moisture and volatiles in the specimens are the three most important results from the Moisture Vapor

Susceptibility Test. Percentage of air voids and aggregate weight per cubic foot are supporting data. All test results are used for comparison with corresponding values from the Relative Stability Test and the trial mixture series.

TABLE IX

RELATIVE STABILITY TEST VALUES FOR THE AVERAGE PERCENTAGE OF
AIR VOIDS, AGGREGATE WEIGHT PER CUBIC FOOT,
AND PERCENTAGE OF MOISTURE AND
VOLATILES IN THE SPECIMENS

Filler	Average Air Voids (%)	Average Aggregate Wt./Ft. ³ (lb/ft ³)	Moisture & Volatiles (%)
No filler	7.1	128.2	0.033
1% hydrated lime	4.5	129.7	0.159
2.5% hydrated lime	6.4	129.8	0.068
4% hydrated lime	8.3	127.6	0.147
2% portland cement	4.6	130.8	0.048
4% portland cement	5.6	132.0	0.076
6% portland cement	5.8	133.5	0.116
2% limestone dust	3.4	130.6	0.015
5% limestone dust	6.0	132.0	0.050
8% limestone dust	9.1	131.0	0.020

Relative Stability Values

Individual relative stability values, the average relative stability values, and the coefficients of variation for the individual values are shown in Table X on page 60. Stability values for the four

control mixtures cannot be compared with the corresponding design values in Table VI on page 47 since the specimens were exposed to moisture vapor before the stability values were obtained.

TABLE X

MOISTURE VAPOR SUSCEPTIBILITY TEST VALUES FOR RELATIVE STABILITY

Filler	Relative Stability Values	Average Relative Stability Value	Coefficient of Variation (%)
No filler	33, 31, 34, 33, 36	33	5.4
1% hydrated lime	29, 33, 32, 32, 26	30	9.5
2.5% hydrated lime	30, 27, 29, 29, 25	28	7.1
4% hydrated lime	28, 29, 25, 33, 28	29	10.1
2% portland cement	19, 14, 20, 17, 12	16	20.5
4% portland cement	27, 17, 17, 25, 20	21	21.7
6% portland cement	34, 28, 36, 37, 32	33	10.7
2% limestone dust	20, 18, 20, 24, 15	19	16.9
5% limestone dust	23, 23, 22, 23, 20	22	5.9
8% limestone dust	42, 45, 44, 46, 51	46	7.4

Air Voids, Aggregate Weight per Cubic Foot, and Moisture and Volatiles

Average values for percentage of air voids, aggregate weight per cubic foot, and percentage of moisture and volatiles in the specimens are contained in Table XI on page 61. Average values for percentage of air voids and aggregate weight per cubic foot for the four control mixtures are reasonably close to the design values in Table VI on page 47 for those particular combinations.

TABLE XI
MOISTURE VAPOR SUSCEPTIBILITY TEST VALUES FOR THE AVERAGE
PERCENTAGE OF AIR VOIDS, AGGREGATE WEIGHT PER
CUBIC FOOT, AND PERCENTAGE OF MOISTURE AND
VOLATILES IN THE SPECIMENS

Filler	Average Air Voids (%)	Average Aggregate Wt./Ft. ³ (lb/ft ³)	Moisture & Volatiles (%)
No filler	6.9	129.1	0.384
1% hydrated lime	5.3	129.7	0.203
2.5% hydrated lime	6.0	130.4	0.424
4% hydrated lime	9.2	127.8	0.529
2% portland cement	4.5	130.7	0.262
4% portland cement	4.8	132.7	0.251
6% portland cement	5.7	134.2	0.346
2% limestone dust	4.2	130.9	0.252
5% limestone dust	4.4	133.2	0.299
8% limestone dust	8.9	131.4	0.393

III. MINNESOTA COLD WATER ABRASION TEST

Specimens having planned asphalt ratios, indicated asphalt ratios, and reduced asphalt contents were all evaluated by the Minnesota Cold Water Abrasion Test. Planned asphalt ratios and indicated asphalt ratios are explained on pages 49 and 50. Reduced asphalt contents are discussed on page 53.

Abrasion Loss

Abrasion loss is the only test result from the Minnesota Cold Water Abrasion Test. It is the loss of material from the

specimens measured as a percentage of the original specimen weight.

Specimens with Planned and Indicated Asphalt Ratios. Abrasion losses for the specimens with planned asphalt ratios are included in Table XII together with the abrasion losses for the specimens with indicated asphalt ratios.

TABLE XII

MINNESOTA COLD WATER ABRASION TEST VALUES FOR ABRASION LOSS FOR
SPECIMENS HAVING PLANNED ASPHALT RATIOS AND FOR
SPECIMENS HAVING INDICATED ASPHALT RATIOS

Filler	Planned Asphalt Ratio (%)	Abrasion Loss (%)	Indicated Asphalt Ratio (%)	Abrasion Loss (%)
No filler	7.0	2.9	7.0	3.5
1% hydrated lime	7.5	3.0	6.8	2.3
2.5% hydrated lime	6.8	2.3	6.8	2.4
4% hydrated lime	6.1	3.0	6.8	3.6
2% portland cement	7.5	2.4	6.8	1.1
4% portland cement	6.8	2.9	6.8	2.3
6% portland cement	6.1	3.2	6.8	1.4
2% limestone dust	7.6	3.7	6.3	2.8
5% limestone dust	6.3	3.5	6.3	1.9
8% limestone dust	5.1	7.0	6.3	3.3

Specimens with Reduced Asphalt Ratios. Abrasion losses for specimens containing no filler with reduced asphalt contents are contained in Table XIII on page 63. Abrasion losses for specimens tested in an earlier pilot study at the University of Idaho Materials Testing Laboratory (4) are shown in Table XIV on page 63.

TABLE XIII

MINNESOTA COLD WATER ABRASION TEST VALUES FOR ABRASION LOSS
FOR SPECIMENS CONTAINING NO FILLER WITH
REDUCED ASPHALT RATIOS

Asphalt Ratio (%)	Abrasion Loss (%)	Asphalt Ratio (%)	Abrasion Loss (%)
6.0	3.8	4.4	11.5
5.0	6.2	4.2	16.6
4.0	18.1	4.0	18.5
3.0	Fell apart when removed from mold	3.8	31.7
		3.6	43.2

TABLE XIV

MINNESOTA COLD WATER ABRASION TEST RESULTS FROM UNPUBLISHED
IDAHO DEPARTMENT OF HIGHWAYS REPORT ENTITLED
"ANALYSIS OF MINERAL FILLER INVESTIGATION
PILOT STUDY"

Filler	Idaho Department of Highways Pit Source		
	Bingham 68 Asphalt Ratio-5.4%	Cassia 129 Asphalt Ratio-5.5%	Clark 27 Asphalt Ratio-5.2%
	Abrasion Loss (%)		
No filler	4.7	3.4	7.0
1% hydrated lime	1.2	3.2	7.9
2% hydrated lime	1.3	6.4	2.5
1% portland cement	2.7	3.1	4.5
2% portland cement	1.5	2.2	4.2
1% Treasurelite	5.2	2.2	10.7
2% Treasurelite	4.8	3.8	7.4
1% limestone dust	2.0	-	5.3
2% limestone dust	1.6	2.7	5.2

IV. IMMERSION-COMPRESSION TEST

Specimens having planned asphalt ratios and specimens having indicated asphalt ratios were both evaluated by the Immersion-Compression Test. Unconfined compression strength for dry specimens, unconfined compression strength for immersed specimens, their corresponding coefficients of variation, and the index of retained strength are the five important test results obtained from the Immersion-Compression Test. Filler-asphalt ratio is supporting data.

Index of retained strength is a relative measure of the reduction in compression strength due to the action of water on the specimen. It is defined as the ratio of the immersed unconfined compression strength to the dry unconfined compression strength of the specimens.

Unconfined Compression Strengths

Planned asphalt ratios are the design asphalt ratios shown in Table VII on page 49 and again in Table XII on page 62. Indicated asphalt ratios are those having the control asphalt ratio for each mineral filler for all three mixture combinations containing that filler as shown in Table XII.

Specimens With Planned Asphalt Ratios. Average unconfined compression strengths for both the dry and immersed specimens are included in Table XV on page 65 together with the index of retained strength for the specimens. The coefficients of variation for the individual values of the dry and immersed unconfined compression strengths are also shown as are the filler-asphalt ratios for the different mixture combinations.

TABLE XV

IMMERSION-COMPRESSION TEST VALUES FOR AVERAGE DRY AND IMMersed UNCONFINED
COMPRESSION STRENGTH AND INDEX OF RETAINED STRENGTH
FOR SPECIMENS HAVING PLANNED
ASPHALT RATIOS

Filler	Dry Specimens		Immersed Specimens		Index of Retained Strength (%)	Filler-Asphalt Ratio
	Average Strength (psi)	Coeff. of Variation (%)	Average Strength (psi)	Coeff. of Variation (%)		
No filler	233	4.3	176	5.2	75.5	-
1% hydrated lime	289	4.5	221	15.6	76.5	0.054
2.5% hydrated lime	327	2.9	277	2.5	84.7	0.146
4% hydrated lime	508	2.5	415	8.2	81.7	0.257
2% portland cement	262	4.7	188	4.8	71.8	0.084
4% portland cement	283	4.7	218	4.7	77.0	0.184
6% portland cement	254	10.2	236	5.3	93.0	0.302
2% limestone dust	304	6.1	249	3.9	81.9	0.095
5% limestone dust	339	5.7	290	6.8	85.5	0.281
8% limestone dust	469	3.2	346	7.5	73.8	0.538
Average Values		4.9		6.5		

Specimens With Indicated Asphalt Ratios. Average unconfined compression strengths for both the dry and immersed specimens are included in Table XVI on page 67 together with the index of retained strength for the specimens. The coefficients of variation for the individual values of the dry and immersed unconfined compression strengths are also shown as are the filler-asphalt ratios for the different mixture combinations.

TABLE XVI

IMMERSION-COMPRESSION TEST VALUES FOR AVERAGE DRY AND IMMERSED UNCONFINED
COMPRESSION STRENGTH AND INDEX OF RETAINED STRENGTH
FOR SPECIMENS HAVING INDICATED
ASPHALT RATIOS

Filler	Dry Specimens		Immersed Specimens		Index of Retained Strength (%)	Filler-Asphalt Ratio
	Average Strength (psi)	Coeff. of Variation (%)	Average Strength (psi)	Coeff. of Variation (%)		
No filler	217	13.6	198	11.0	91.2	-
1% hydrated lime	264	1.4	219	28.9	83.0	0.059
2.5% hydrated lime	299	10.1	308	6.8	103.0	0.146
4% hydrated lime	346	4.3	392	4.5	113.0	0.231
2% portland cement	264	0.8	207	4.0	78.4	0.094
4% portland cement	278	4.6	204	7.4	73.3	0.184
6% portland cement	271	3.3	240	3.5	88.5	0.270
2% limestone dust	235	3.6	179	1.3	76.3	0.115
5% limestone dust	279	5.1	214	12.4	76.7	0.281
8% limestone dust	370	20.1	353	26.0	95.4	0.435
Average Values		6.7		10.6		

CHAPTER V

ANALYSIS OF TEST RESULTS

Effect of temperature control on specimens, optimum asphalt ratios, and planned asphalt ratios are discussed to point out certain problems or trends in those areas. Following that, each of the four tests is reviewed individually to examine pertinent test results and appraise their value. The analysis is completed by comparing the different test values to determine significant relationships and evaluate their importance.

An upper limit has been suggested for the coefficient of variation for relative stability values and also for the coefficient of variation for dry and immersed unconfined compression strength values. Each of the suggested upper limits is the higher confidence limit at the 95 per cent confidence level for the mean of the population of coefficients of variation for the test values under consideration.

Duncan's New Multiple-Range Test has been introduced to help analyze results from the Relative Stability Test, the Moisture Vapor Susceptibility Test, and the Immersion-Compression Test. Duncan's New Multiple-Range Test uses multiple comparisons to examine each mean test value for significant difference from other mean test values from the same test or similar type test (46). It has been used in this study to determine:

1. which average relative stability values from the Relative Stability and Moisture Vapor Susceptibility Tests

are significantly different from other average values from those two tests,

2. which average dry and immersed compression strength values from the Immersion-Compression Test are significantly different from other average values from that test.

These comparisons are shown in Appendix D on page 137.

Linear correlation coefficient is a measure of the degree to which variables vary together or a measure of the intensity of association (46). It has been used in this investigation to examine the possibility of predicting the results for one test from the results of a different test. Linear correlation coefficients for the different test values are contained in Appendix D on page 139.

I. TRIAL MIXTURE SPECIMENS

Certain problems and trends became apparent upon development of the trial mixture specimens. These are discussed in the following two sections.

Effect of Temperature Control

Results of the auxiliary investigation did not indicate that temperature control would improve repeatability for values or relative stability, mixture weight per cubic foot, and percentage of air voids obtained by performing a trial mixture series several times. Coefficients of variation shown in Table V on page 44 vary as widely for values obtained with temperature control as for similar values obtained without temperature control. However, the erratic results may have

been caused by the method by which temperature control was accomplished. Once the specimen was compacted, a thermistor probe was attached to the top of the specimen to record the temperature. The probe was kept in place by tape since drilling a hole in the specimen would have affected the stability value. It is questionable whether the probe was able to record the specimen temperature with the required degree of accuracy. Further investigation in this area will require the use of a more positive means of temperature control such as a forced draft constant temperature type oven.

Optimum Asphalt Ratios

Relative stability value and the slope of the stability curve were the controlling factors in selecting the optimum asphalt ratios for all mixture combinations except those containing no filler, 1 per cent hydrated lime, and 2 per cent portland cement. These three combinations were limited to 7.0 per cent asphalt since mixtures with greater asphalt contents are used only when required by special conditions. Increasing the amount of mineral filler reduced the optimum asphalt ratio each time for all three types of filler. This trend is similar to one in another investigation (29) which concluded in part that "if the filler content is increased, the amount of asphaltic cement will not be increased but rather will be reduced."

II. PLANNED ASPHALT RATIOS

Planned asphalt ratios for mixtures containing the same type of filler did not have as great a variation from their respective

optimum asphalt ratios as had originally been anticipated. This was due primarily to the reduction in optimum asphalt ratio with increasing amount of filler. It had been anticipated that increasing the amount of filler would increase the required asphalt content. As such was not the case, the planned asphalt ratios were within 0.5 per cent to 0.8 per cent of the corresponding optimum asphalt ratio for five of the six different mixture combinations and was exactly the same for 6 per cent portland cement. Thus, while the variation in filler-asphalt ratio was quite satisfactory, the variation in planned asphalt ratio from optimum asphalt ratio was not as great as desired. However, the variation was large enough to produce mixture designs from which low or marginal values were obtained for portland cement and limestone dust specimens for three of the tests. This permitted a good comparison to be made between the sensitivities of the different tests.

III. RELATIVE STABILITY TEST

The average relative stability value of 45 for no filler indicates that a mineral filler is not needed to improve the stability value of asphalt mixtures using aggregate from Pit Source Ada 53. Furthermore, it is evident from Tables VI and VIII on pages 47 and 58 that addition of a filler reduces the relative stability value for most of the mixture combinations used. Only the 2 per cent portland cement mixture combination at optimum asphalt content provides any improvement in relative stability value.

When fillers were used, increasing the amount resulted in a higher stability value since the increase in filler was accompanied

by a decrease in asphalt content. This results in an increase in internal friction of the specimen and a higher stability value. This trend is quite evident for both the portland cement and limestone dust mixture combinations.

The average coefficient of variation for the individual relative stability values is 12.2 per cent. Two of the mixture combinations, 1 per cent hydrated lime and 2 per cent portland cement, have very large coefficients of variation. This may be due to the exceptionally high asphalt content of 7.5 per cent in those two combinations. Since this test procedure was designed to simulate routine laboratory testing, moderate variations are to be expected in individual test values. Nevertheless, examination of the ranges of individual values contained in Table VIII on page 58 indicates that the maximum coefficient of variation for individual values should be 15 per cent. That percentage is the higher confidence limit at the 95 per cent confidence level for the population mean of those coefficients of variation.

With the exception of the mixture combinations for portland cement, it was not possible to maintain the desired constant percentage of air voids in specimens having the same type of filler. As previously stated, the mixtures containing 4 per cent hydrated lime and 8 per cent limestone dust could not be compressed to the same volume as the other specimens with the same type of filler. Thus the percentage of air voids in those two series of mixtures are considerably higher than the values for the rest of the specimens containing the same

type of filler. However, it is not believed that this variation substantially affected any of the test values.

IV. MOISTURE VAPOR SUSCEPTIBILITY TEST

The average relative stability value of 33 for no filler indicates that a mineral filler is still not needed in asphalt mixtures using aggregate from Pit Source Ada 53 if the minimum value of 30 established by the California Division of Highways is used. Again the presence of filler reduces the relative stability value for most mixture combinations with only 8 per cent limestone dust providing improvement in relative stability value.

Increasing the amount of filler resulted in higher relative stability values as was the case for the Relative Stability Test. This trend is again quite noticeable for mixtures containing portland cement or limestone dust.

The average coefficient of variation for the individual relative stability values is 11.2 per cent which is slightly lower than the average coefficient of variation of 12.2 per cent for individual stability values for the Relative Stability Test. The small reduction may be due to natural variation or it may be due to the additional curing the specimens underwent during the period of exposure to moisture vapor. However, three of the mixture combinations still have large coefficients of variation. Examination of the ranges of individual values in Table X on page 60 still indicates that 15 per cent should be a maximum for coefficient of variation for individual values. The recommended limit

of 15 per cent is again the higher confidence limit at the 95 per cent confidence level for the population mean of those coefficients of variation.

With the exception of the mixture combinations for 4 per cent hydrated lime and 8 per cent limestone dust, it was possible to maintain reasonably constant percentages of air voids in the specimens having the same type of filler thereby permitting development of reasonably constant total volume specimens. This may have been due in part to improved familiarity with the procedure for making the specimens and in part to greater uniformity in aggregate size and shape.

The increase in total percentage of moisture and volatiles in the specimen from that of the Relative Stability Test ranges from 0.044 per cent for 1 per cent hydrated lime to 0.382 per cent for 4 per cent hydrated lime. However, with the exception of 1 per cent hydrated lime and 4 per cent portland cement, the increase is quite consistent ranging from 0.214 per cent to 0.382 per cent with an average increase in total percentage of 0.299 per cent. Such an increase was expected since five of ten average relative stability values from the Moisture Vapor Susceptibility Test were significantly lower than the corresponding values from the Relative Stability Test. However, a maximum permissible increase in moisture content has not as yet been established by any agency.

V. MINNESOTA COLD WATER ABRASION TEST

With the exception of the planned asphalt ratio mixture combination for 8 per cent limestone dust, the abrasion losses for specimens

having planned asphalt ratios range from 2.3 per cent to 3.7 per cent while those for specimens having indicated asphalt ratios range from 1.1 per cent to 3.6 per cent. The 7.0 per cent abrasion loss by the 8 per cent limestone dust specimens could be due merely to a weak edge on just one of the eight specimens. These results are consistent with those from previous tests (4) shown in Table XIV on page 63 which range from 1.2 per cent to 10.7 per cent for specimens containing four different fillers and three different aggregates. With just five exceptions out of 26 test results, the abrasion losses from the previous tests range from 1.2 per cent to 5.3 per cent.

Results of the tests using specimens with no mineral filler and reduced asphalt contents as shown in Table XIII on page 63 indicate that the asphalt ratio must be reduced below 5.0 per cent before any appreciable abrasion loss occurs. This is further supported by study of the original paper (28) from which the test was developed. Asphalt ratios used in that investigation range from 3.75 per cent to 5.3 per cent. However, of the 150 different mixture combinations tested, 135 had asphalt ratios in the range of 4.0 per cent to 5.0 per cent. Thus the test was developed using specimens with lower asphalt contents than are normally used in asphalt surface courses constructed today.

VI. IMMERSION-COMPRESSION TEST

Average dry and immersed unconfined compression strengths of 233 psi and 176 psi, respectively, for planned asphalt ratio specimens with no filler shown in Table XV on page 65 indicate that a mineral

filler would be needed to improve the dry and immersed compression strengths to the present Idaho Department of Highways minimum acceptable values of 250 psi and 213 psi, respectively. Corresponding indicated asphalt ratio strengths from Table XVI on page 67 show the same need. Furthermore, it is evident from the values in these two tables that all but one mixture combination provides an improvement in both dry and immersed strength. Only the immersed strength for the 2 per cent limestone dust mixture combination with an indicated asphalt ratio failed to show improvement.

Increasing the amount of filler resulted in increasingly higher dry and immersed strengths. This is the result of greater internal cohesion in the specimen caused by the reaction between the filler and the asphalt. This trend is quite evident for both the hydrated lime and limestone dust mixture combinations.

The average coefficient of variation for the individual strength values is 4.9 per cent for the dry planned asphalt ratio specimens, 6.5 per cent for the immersed planned asphalt ratio specimens, 6.7 per cent for the dry indicated asphalt ratio specimens, and 10.6 per cent for the immersed indicated asphalt ratio specimens. However, the average coefficient of variation for all individual strength values is 7.2 per cent with only seven mixture combinations having coefficients of variation appreciably over 10.0 per cent. Since this test procedure was also designed to simulate routine laboratory testing, moderate variations are again to be expected in individual test values. Thus, examination of Tables XV and XVI on pages 65 and 67 shows that the

maximum coefficient of variation for individual values should be 10 per cent which is the higher confidence limit at the 95 per cent confidence level for the population mean of those coefficients of variation.

Planned asphalt ratios are the same as the indicated asphalt ratios for no filler, 2.5 per cent hydrated lime, 4 per cent portland cement, and 5 per cent limestone dust mixture combinations as can be seen in Table XII on page 62. Hence it was not expected that either the dry or the immersed unconfined compression strengths for specimens having those particular planned asphalt ratios would be significantly different from the corresponding dry or immersed strengths for specimens having those indicated asphalt ratios. However, examination of Duncan's New Multiple-Range Test in Appendix D on page 138 shows that the average strength values of the dry and immersed specimens for 5 per cent limestone dust in Table XV on page 65 are significantly different from the corresponding values in Table XVI on page 67. This points out that in specimens with identical amounts of the same components there can be a significant difference in cohesion because of particle size, shape, and distribution within the mixture. Shape, position, and distribution of the larger particles of aggregate seem to have a substantial effect on the compression strength of the specimen, especially when it contains a high percentage of mineral filler. Thus significant differences in strength values can be expected in specimens with identical amounts of the same components.

VII. COMPARISON OF TESTS

Different combinations of tests are compared to determine the intensity of association of relationships and appraise their importance. Four such combinations are evaluated to establish the basis for development of the proposed standard test procedure.

Relative Stability and Moisture Vapor Susceptibility Tests

Relative stability values for the two tests have been plotted in Figure 5 on page 79. The linear correlation coefficient for these two sets of values is 0.79 which is only fair. However, the values for 8 per cent limestone dust cause a reduction of several per cent in the value of the correlation coefficient.

Figure 6 on page 80 is, in part, a plot of the relative stability values for the two tests against their filler-asphalt ratios.

This comparison shows:

1. good sensitivity of the Relative Stability Test to variation in filler-asphalt ratio,
2. fairly good sensitivity of the Moisture Vapor Susceptibility Test to variation in filler-asphalt ratio,
3. only fair sensitivity of the Moisture Vapor Susceptibility Test to the effect of moisture vapor and/or water on the specimens.

The linear correlation coefficient for the two sets of values for percentage of moisture and volatiles in the specimen is 0.11 which is very poor. No information of value is gained from plotting those values either by themselves or against the filler-asphalt ratios.

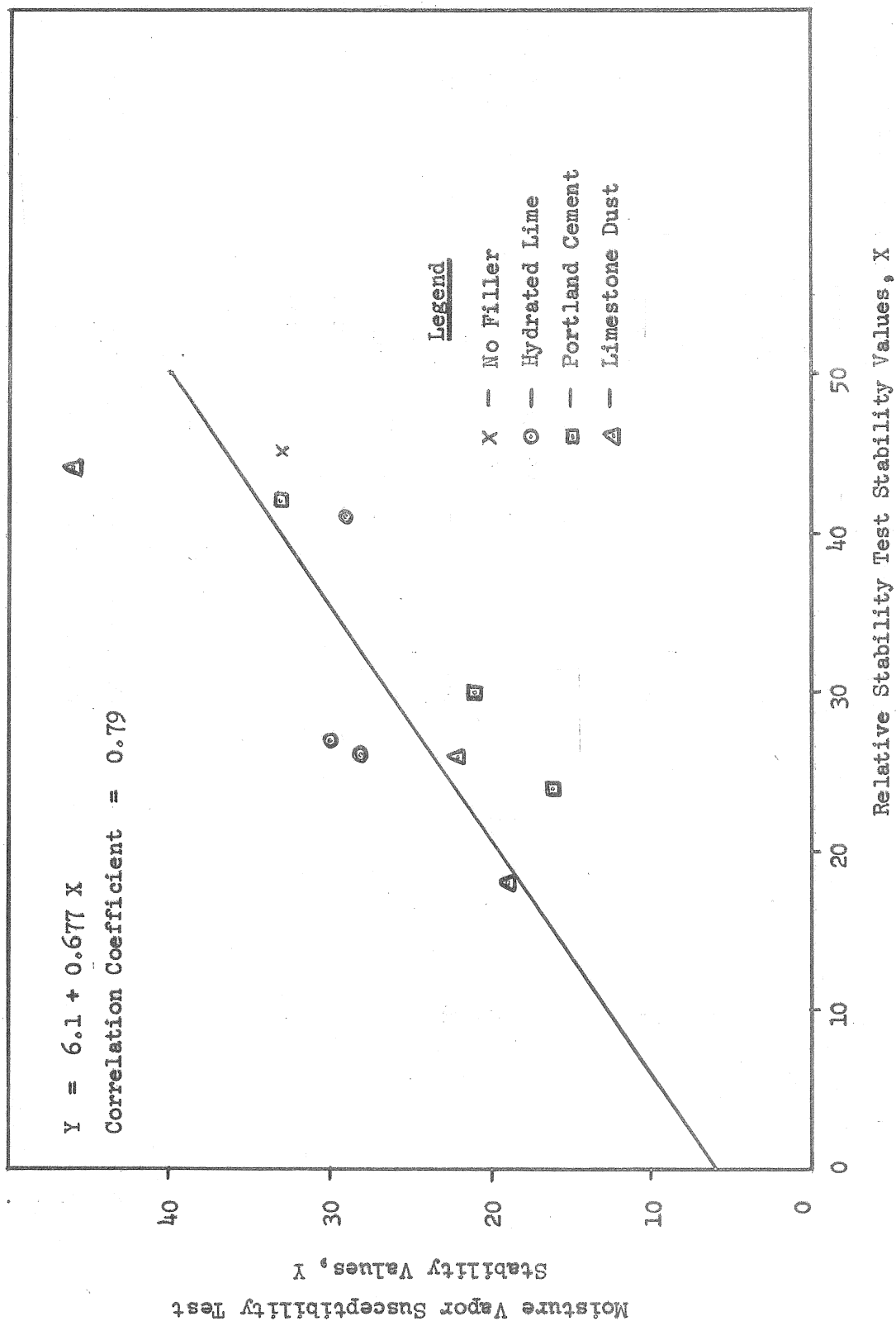


Figure 5. Relative Stability Test Relative Stability vs Moisture Vapor Susceptibility Test Relative Stability

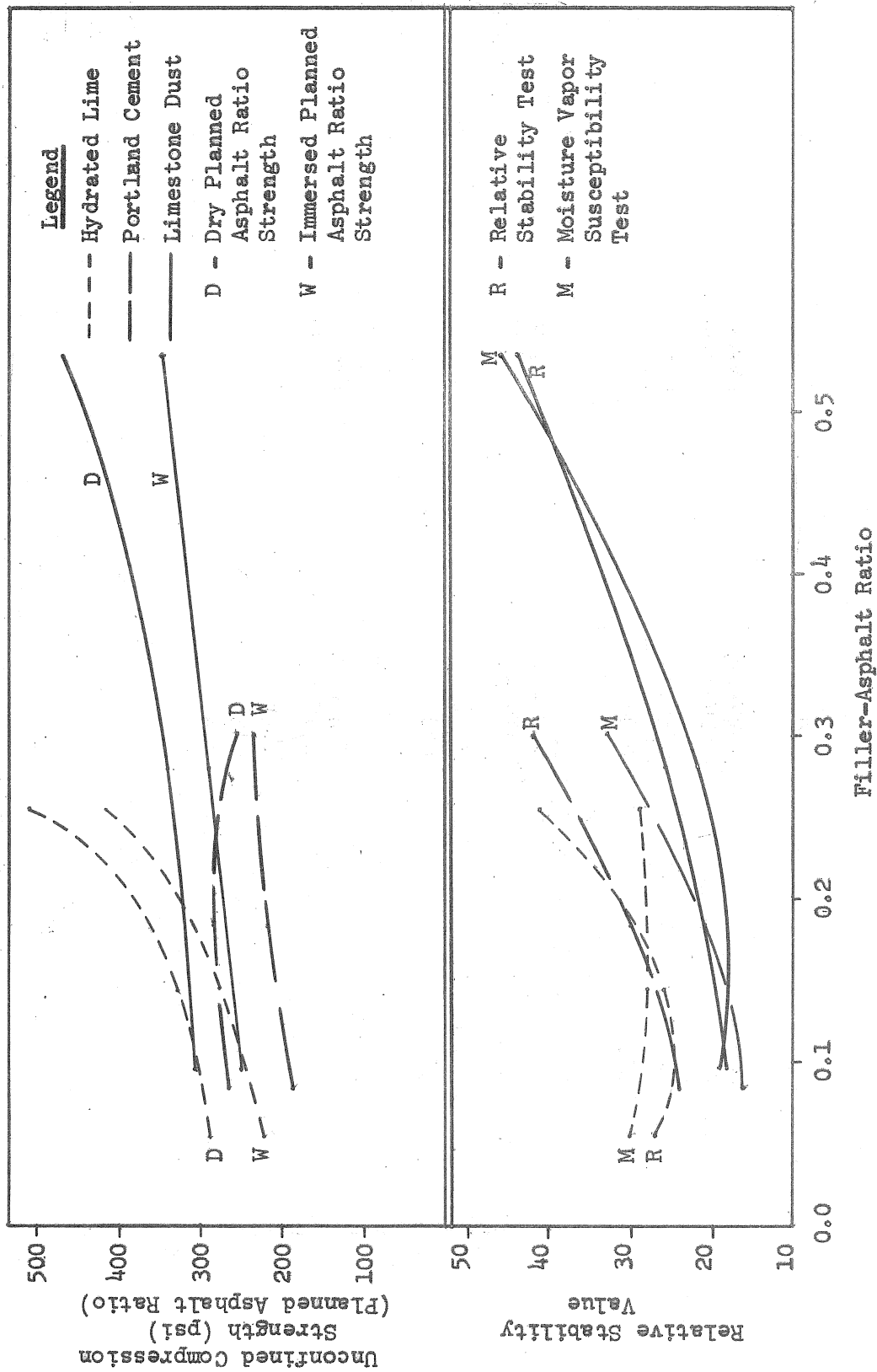


Figure 6. Relative Stability Values and Unconfined Compression Strength (Planned Asphalt Ratio) vs Filler-Asphalt Ratio

Relative Stability, Moisture Vapor Susceptibility and Minnesota Cold Water Abrasion Tests

Linear correlation coefficients for both sets of relative stability values plotted against the abrasion loss values are 0.37 and 0.69 which are very poor. No information of value can be gained from these plots or by plotting the abrasion loss values against the filler-asphalt ratios.

Relative Stability, Moisture Vapor Susceptibility and Immersion-Compression Tests

Figure 6 on page 80 also shows the dry and immersed compression strengths for the planned asphalt ratio specimens plotted against the filler-asphalt ratios together with the previously mentioned plot of the two sets of relative stability values against the filler-asphalt ratios. This combination plot shows:

1. good sensitivity of the Immersion-Compression Test to variation in filler-asphalt ratio,
2. good sensitivity of the Immersion-Compression Test to the loss of cohesion resulting from the action of water on the specimens.

Dry and immersed unconfined compression strengths for planned asphalt ratio specimens have been plotted in Figure 7 on page 82. The linear correlation coefficient for these two sets of values is 0.97 which is excellent. Figure 7 also shows the plot of the dry strength for the indicated asphalt ratio specimens against the immersed strength. The linear correlation coefficient for these two sets of values is 0.84 which is fairly good.

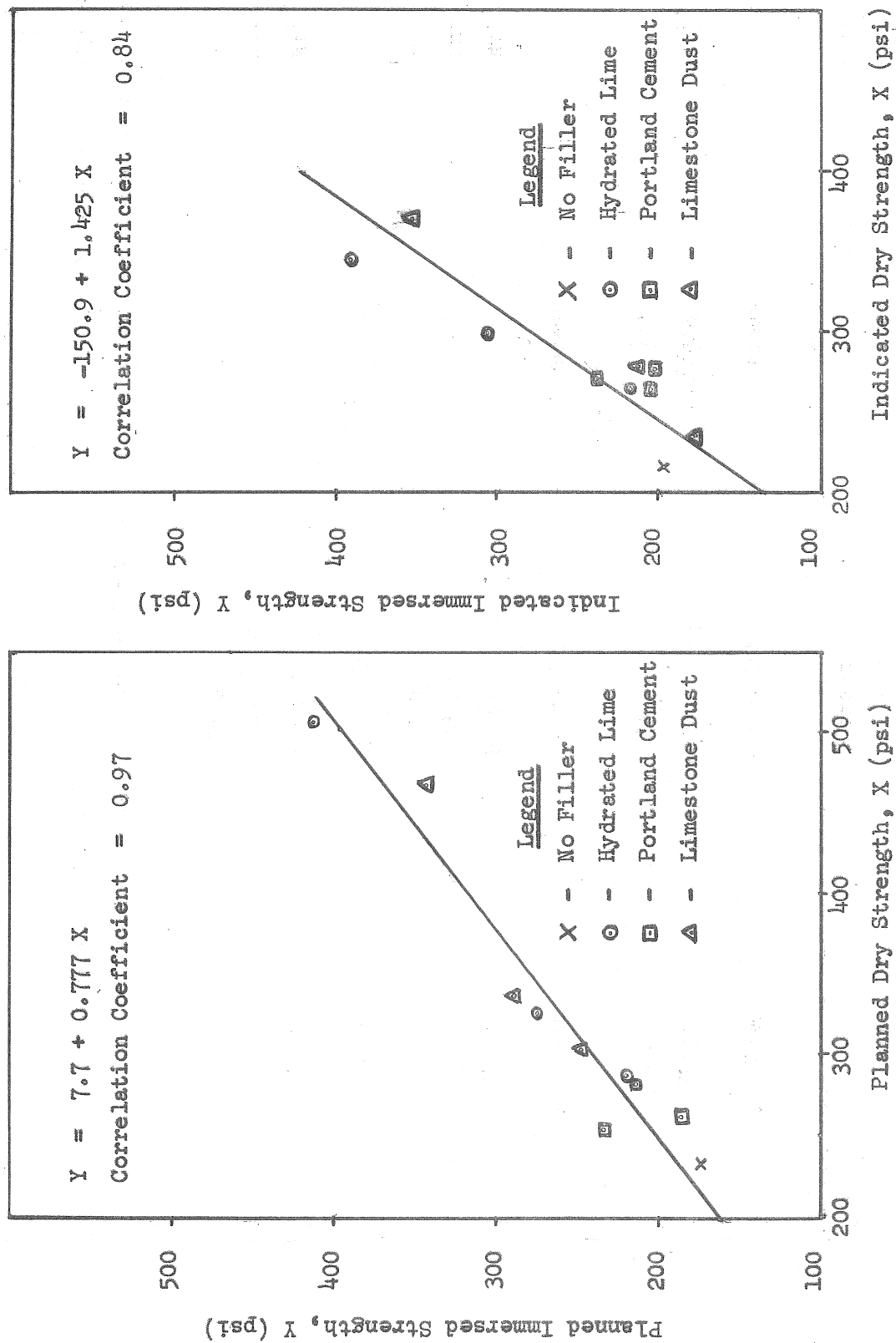


Figure 7. Planned and Indicated Dry vs Immersed Unconfined Compression Strength

Figure 8 on page 84 shows the dry and immersed strengths for the indicated asphalt ratio specimens plotted against the filler-asphalt ratios together with the same plot of the two sets of relative stability values against the filler-asphalt ratios. This combination plot shows:

1. continued good sensitivity of the Immersion-Compression Test to variation in filler-asphalt ratio,
2. fairly good sensitivity of the Immersion-Compression Test to the loss of cohesion by the specimens from water action.

The change in trend evidenced by the hydrated lime specimen strengths is attributed to variation in shape, position and distribution of the larger aggregate particles within the mixture as previously discussed.

Minnesota Cold Water Abrasion and Immersion-Compression Tests

Linear correlation coefficients for the dry and immersed unconfined compression strengths for planned asphalt ratio specimens plotted against the abrasion loss values are 0.54 and 0.43, respectively, which are very poor. No information of value can be gained from these plots or by plotting the abrasion loss values against the filler-asphalt ratios.

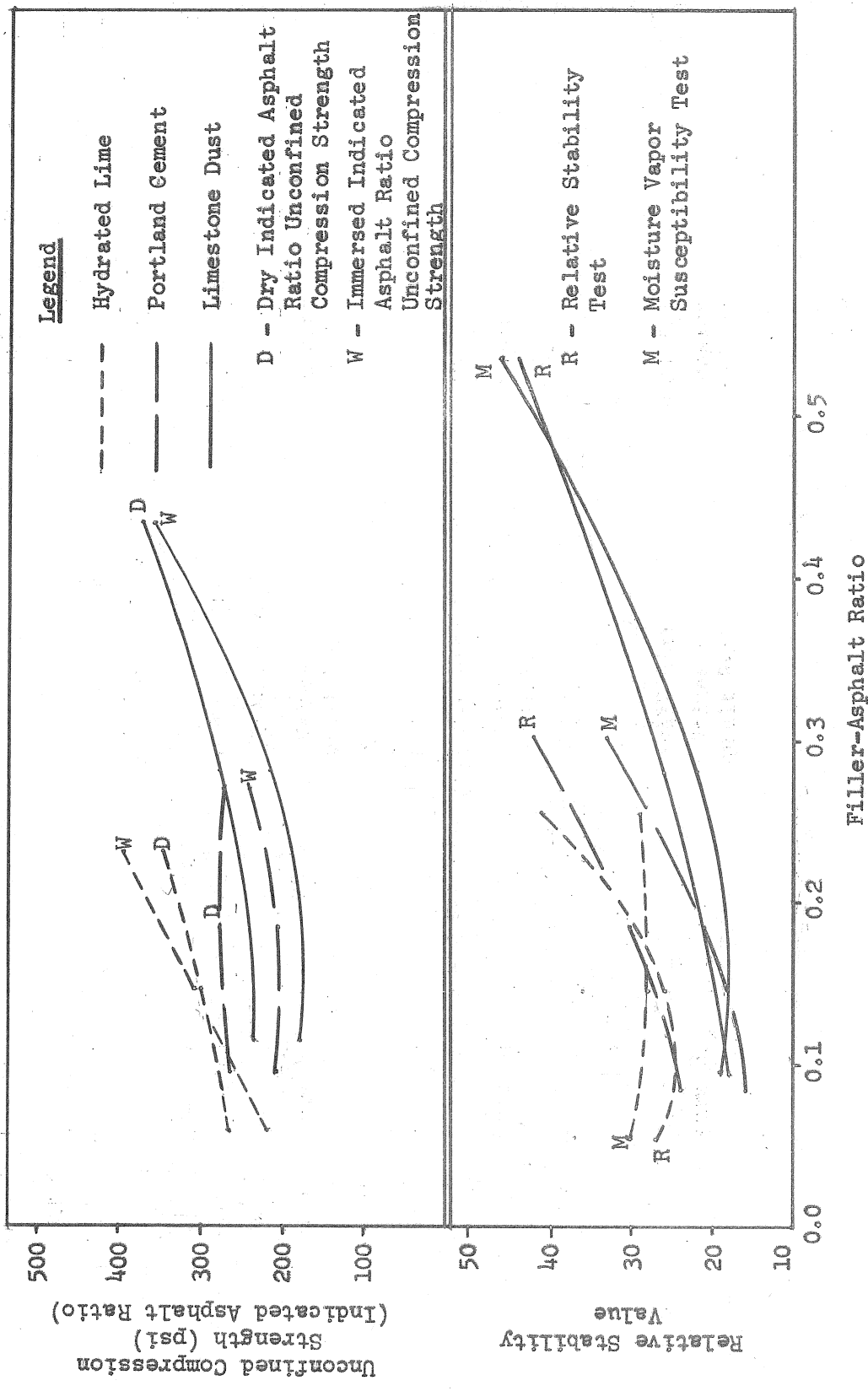


Figure 8. Relative Stability Values and Unconfined Compression Strength (Indicated Asphalt Ratio) vs Filler-Asphalt Ratio

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

Conclusions contained in this study are those reached after analyzing the physical test results and the related pertinent literature. Recommendations, however, are based not only on the conclusions but also on knowledge gained from past experience which has been made more meaningful by this investigation.

I. CONCLUSIONS

The following conclusions are the result of a review of pertinent literature, an examination of the method of investigation, and the analysis of the test results.

1. Proportioning the constituents of an asphalt mixture by solid volume rather than by weight permits more accurate control of the resulting percentage of air voids in the mixture. By knowing the individual solid volumes of the constituents, it is possible to maintain a reasonably constant percentage of air voids in specimens compressed to a constant total volume.
2. Use of a constant total volume of air voids permits better comparison between specimens containing different mineral fillers. This approach, which has been described in detail on pages 30 and 31, minimizes effects due to

variation in air void content on test results.

3. Trial asphalt ratios over 6.0 per cent should be selected at 0.5 per cent intervals up to 7.5 per cent to obtain additional values of relative stability in that critical range of asphalt ratios.
4. Optimum asphalt ratio should be selected such that the corresponding relative stability value is located on a part of the stability curve where the rate of change of the slope is small. This provides the greatest probability of being able to obtain good repeatability of relative stability values.
5. Mixing, curing, and compaction temperatures of all specimens should be carefully controlled throughout investigations of this type using a positive means of temperature control such as a constant temperature mechanical convection-type oven. Mixing, curing, and compaction temperatures should be 300° F., 230° F., and 230° F., respectively, in accordance with Idaho T-9-58 and Idaho T-25-64.
6. Increasing the amount of mineral filler in a mixture can decrease the optimum asphalt ratio for the mixture as shown in Table VI on page 47.
7. The filler-asphalt ratio, which has been used by other investigators (11) (15), is a valuable criteria for analysis in that it provides a standard base for evaluating the effects of fillers on the different physical

properties of mixtures. This is illustrated in Figure 6 on page 80 and Figure 8 on page 84 where the filler-asphalt ratio has been used to help determine the sensitivity of the Immersion-Compression Test.

8. The Relative Stability Test

- a) indicates that a mineral filler does not materially improve relative stability values of asphalt mixtures using aggregate from Pit Source Ada 53 as shown in Table VIII on page 58,
- b) shows good sensitivity to variation in filler-asphalt ratio as illustrated in Figure 6 on page 80.

9. The Moisture Vapor Susceptibility Test

- a) indicates that a mineral filler is not required in asphalt mixtures using aggregate from Pit Source Ada 53 if the California Highway Department minimum standard of 30 is used,
- b) shows fairly good sensitivity to variation in filler-asphalt ratio as shown in Figure 6 on page 80,
- c) shows only fair sensitivity to the effect of moisture vapor and water on the specimens as also illustrated in Figure 6,
- d) shows no consistent relationship between the relative stability values and the values for percentage of moisture and volatiles in the specimen as is evident by the linear correlation coefficient of 0.47.

10. The maximum coefficient of variation for individual values of relative stability for specimens containing identical amounts of the same constituents should be 15 per cent for the reasons given on pages 72 to 74.
11. The Minnesota Cold Water Abrasion Test
 - a) shows in Table XIII on page 63 that the asphalt ratio must be reduced below 5 per cent before any appreciable abrasion loss occurs,
 - b) was developed as discussed on page 75 using specimens with lower asphalt contents than are normally used in present-day construction of asphalt surface courses,
 - c) does not provide meaningful results when higher asphalt contents are used.
12. The Immersion-Compression Test
 - a) indicates that a mineral filler would improve asphalt mixtures using aggregate from Pit Source Ada 53 as can be seen in Tables XV and XVI on pages 65 and 67, respectively,
 - b) shows good sensitivity to variation in filler-asphalt ratio as illustrated in Figures 6 and 8 on pages 80 and 84, respectively,
 - c) shows good sensitivity to the loss of cohesion by the specimens from water action as can also be seen in Figures 6 and 8.

13. The maximum coefficient of variation for individual values of unconfined compression strength for specimens containing identical amounts of the same constituents should be 10 per cent for the reasons given on pages 76 and 77.
14. The 24-hour immersion period at 140° F. in the Immersion-Compression Test seems to be adequate. However, further investigations in this area should also examine the 4-day immersion period at 120° F. for purposes of comparison to prove the adequacy of the 24-hour immersion period at 140° F.
15. There can be a significant difference in the strength values of Immersion-Compression Test specimens with identical amounts of the same components if there is variation in size, shape, and distribution of the particles. These factors seem to have a pronounced effect on the cohesion of the specimen especially when it contains a high percentage of mineral filler as discussed on page 77.
16. Each mineral filler effects an asphalt aggregate mixture in a little different way. This is evident from the variation in values for relative stability and unconfined compression strength resulting from the different mixture combinations.
17. Some mineral fillers seem to be more effective than others in improving desirable characteristics of an asphalt mixture. This investigation indicates that for mixtures

using Pit Source Ada 53 aggregate, hydrated lime is the most effective filler with portland cement and limestone dust providing less improvement.

18. As the Immersion-Compression Test measures primarily cohesion and the Relative Stability Test measures principally internal friction of the specimen, both tests are required to determine in part the resistance of an asphalt mixture to failure from asphalt absorption or stripping.

II. RECOMMENATIONS

The following recommendations are based upon analysis of the method of investigation, the test results, the conclusions, and past experience which is now of significant value because of this study:

1. The standard test method should be the use of the Relative Stability Test and the Immersion-Compression Test for evaluating the improvement in resistance to deformation, resistance to water action, cohesion, and strength of asphalt mixtures through use of a mineral filler.
2. The 24-hour immersion period at 140° F. in the Immersion-Compression Test should be tentatively established as standard procedure. Use of that immersion period will require final substantiation by comparison to the 4-day immersion period at 120° F. during Phase Two of this study.

3. The Moisture Vapor Susceptibility Test should be further analyzed after the procedure has been modified to more nearly duplicate conditions under which asphalt stripping occurs in the field. A method needs to be found by which the mixture can be subjected to repeated stress at varying temperatures while it is being exposed to moisture vapor. This will permit a more realistic evaluation of mixture resistance to stripping action.
4. The Minnesota Cold Water Abrasion Test should be dropped from further consideration as it does not provide meaningful results at the higher asphalt contents being used in present-day asphalt surface course construction.
5. The procedure of proportioning the constituents of an asphalt mixture by volume rather than by weight should be investigated further.
6. Asphalt ratios for trial mixture series specimens should be selected at 4.0%, 5.0%, 6.0%, 6.5%, 7.0%, and 7.5% to obtain additional values of relative stability in the range of asphalt ratios over 6.0 per cent.
7. Percentage of air voids in asphalt mixtures should be determined using the maximum specific gravity (Rice Method). Specimen volumes should be computed by weighing the specimen originally in air, then in water and then again in air to correct for absorption of water.

8. Filler-asphalt ratio should be used as the basic measurement of constituent proportion in the second phase of this study.
9. During Phase Two of this investigation the following two methods of adding mineral filler to a mixture should be examined:
 - a) add the filler to the dry aggregate, mix, and then add the asphalt. The principal reaction would thus be between the filler and the asphalt as was the case in this investigation,
 - b) add hydrated lime to the dampened aggregate, allow the mixture to cure for 48 hours, and then add the asphalt. The principal reaction would thus be between the hydrated lime and the dampened aggregate.
10. SM-K cationic emulsion asphalt should be evaluated during Phase Two of this investigation. Should its use in mixtures with mineral filler prove unfeasible, two different brands of 85-100 penetration asphalt cement should be examined to determine if different brands of asphalt produce significant differences in test results.
11. Aggregate from five different pit sources having a history of absorption and stripping problems should be evaluated during Phase Two of this study. Pit sources Bonner 46, Clearwater 36-s, Idaho 93, Oneida 44 and Twin Falls 63 are recommended as appropriate for investigation.

REFERENCES CITED

REFERENCES CITED

1. Howard, T. R., Availability of Aggregates in Southern District II, Research Project No. 34, Idaho Department of Highways, March, 1965.
2. Eager, William L., "Effect of Moisture on Bituminous Pavements in Rocky Mountain Areas," Highway Research Board Record, 51:100-111, 1964.
3. Traxler, R. N., F. R. Olmstead, and R. E. Bollen, "Comparison of the Various Tests Used to Evaluate Mineral Fillers," Proceedings of the Association of Asphalt Paving Technologists, 9:218-256, 1937.
4. Sylvies, William A., "Analysis of Mineral Filler Investigation Pilot Study," Unpublished Idaho Department of Highways report, November, 1965.
5. Tunnickliff, David G., "A Review of Mineral Filler," Proceedings of the Association of Asphalt Paving Technologists, 31:118-150, 1962.
6. Richardson, Clifford, The Modern Asphalt Pavement, John Wiley & Sons, New York, 1905.
7. Miller, J. S., Jr., and R. N. Traxler, "Fundamental Physical Properties of Mineral Fillers - I," Proceedings of the Association of Asphalt Paving Technologists, 4:75-83, 1932.
8. Traxler, R. N., and J. S. Miller, Jr., "Mineral Powders, Their Physical Properties and Stabilizing Effects," Proceedings of the Association of Asphalt Paving Technologists, 7:112-123, 1936.
9. Traxler, R. N., "The Evaluation of Mineral Powders as Fillers for Asphalt," Proceedings of the Association of Asphalt Paving Technologists, 8:60-67, 1937.
10. Bollen, R. E., "The Selection and Use of Mineral Fillers for Low Cost Roads," Proceedings of the Association of Asphalt Paving Technologists, 8:39-59, 1937.
11. Warden, W. B., S. B. Hudson, and H. C. Howell, "Evaluation of Mineral Fillers in Terms of Practical Pavement Performance," Proceedings of the Association of Asphalt Paving Technologists, 28:316-352, 1959.
12. Kallas, B. F., and H. C. Krieger, "Effects of Consistency of Asphalt Cements and Type of Mineral Filler on the Compaction of Asphalt Concrete," Proceedings of the Association of Asphalt Paving Technologists, 29:152-176, 1960.

13. Kallas, B. F., and V. P. Puzinauskas, "A Study of Mineral Fillers in Asphalt Paving Mixtures," Proceedings of the Association of Asphalt Paving Technologists, 30:493-528, 1961.
14. Csanyi, Ladis H., "Functions of Fillers in Bituminous Mixes," Highway Research Board Bulletin, 329:1-5, 1962.
15. Kallas, B. F., V. P. Puzinauskas, and H. C. Krieger, "Mineral Fillers in Asphalt Paving Mixtures," Highway Research Board Bulletin, 329:6-29, 1962.
16. Csanyi, Ladis H., Rodney E. Cox, and Charles R. Teagle, "Effect of Fillers on Asphaltic Concrete Mixes," Highway Research Board Record, 51:68-88, 1964.
17. Lowrie, Charles R., "Use of Hydrated Lime in Highway Construction," Colorado Highway Department. A paper presented at the Thirty-Seventh Annual University of Colorado Highway Engineer's Conference, February 20, 1964.
18. Heukelom, W., "The Role of Filler in Bituminous Mixes," Proceedings of the Association of Asphalt Paving Technologists, 34:396-429, 1965.
19. Hveem, F. N., and B. A. Vallerger, "Density Versus Stability," Proceedings of the Association of Asphalt Paving Technologists, 21:237-262, 1952.
20. McCarty, L. E., "Correlation between Stability and Certain Physical Properties of Bituminous Materials," Highway Research Board Proceedings, 33:322-340, 1954.
21. Griffith, John M., and B. F. Kallas, "Influence of Fine Aggregate on Asphaltic Concrete Paving Mixtures," Highway Research Board Proceedings, 37:219-255, 1958.
22. Minor, Carl E., "Correlation of Hveem Stabilometer and Cohesimeter Test Results and Kneading Compactor Densities with Service Records of Bituminous Pavements," American Society of Testing Materials, Special Technical Publication No. 252:70-80, June, 1959.
23. Skog, J., and E. Zube, "New Test Methods for Studying the Effect of Water Action on Bituminous Mixtures," Proceedings of the Association of Asphalt Paving Technologists, 32:380-411, 1963.
24. Kari, W. J., "Asphalt Treated Bases," American Bitumals and Asphalt Company Technical Paper No. 105. Presented at the Fifth Annual Highway Conference, University of the Pacific, Stockton, California, March 7, 1962.

25. Carpenter, Carl A., "A Cooperative Study of Fillers in Asphaltic Concrete," Public Roads, Vol. 27, No. 5, pp. 101-110, 1952.
26. Goldbeck, A. T., "Immersion-Compression Tests Compared with Laboratory Traffic Tests on Bituminous Concrete," American Society of Testing Materials, Special Technical Publication No. 94:53-66, 1949.
27. Goode, Joseph F., "Use of the Immersion-Compression Test in Evaluating and Designing Bituminous Paving Mixtures," American Society of Testing Materials, Special Technical Publication No. 252:113-129, 1959.
28. Swanberg, John H., and W. L. Hinderman, "The Use of an Abrasion Test as a Measure of Durability of Bituminous Mixtures," American Society of Testing Materials, Special Technical Publication No. 94:67-83, 1949.
29. Kampf, Leo, and William Raisch, "A Study of Voids in Asphalt Paving Mixtures," Proceedings of the Association of Asphalt Paving Technologists, 16:174-202, 1947.
30. Ricketts, W. C. and Others, "An Evaluation of the Specific Gravity of Aggregates for Use in Bituminous Mixtures," Proceedings of the American Society of Testing Materials, 54:1246-1257, 1954.
31. Benson, Fred J., and Bh. Subbaraju, "Specific Gravity of Aggregates in Asphaltic Paving Mixtures," Highway Research Board Proceedings, 34:320-328, 1955.
32. McLeod, Norman W., "Relationships Between Density, Bitumen Content, and Voids Properties of Compacted Bituminous Paving Mixtures," Highway Research Board Proceedings, 35:327-404, 1956.
33. McLeod, Norman W., "Selecting the Aggregate Specific Gravity for Bituminous Paving Mixtures," Highway Research Board Proceedings, 36:282-317, 1957.
34. McLeod, Norman W., "Void Requirements for Dense-Graded Bituminous Paving Mixtures," American Society of Testing Materials, Special Technical Publication No. 252:81-112, 1959.
35. Kiefer, Ralph W., "The Effect of Compaction Temperature on the Properties of Bituminous Concrete," American Society of Testing Materials, Special Technical Publication No. 294:19-30, 1960.
36. Bahri, Gandharv R., and Lloyd F. Rader, "Effects of Asphalt Viscosity on Physical Properties of Asphaltic Concrete," Highway Research Board Record, 67:59-83, 1964.

37. The Asphalt Handbook, The Asphalt Institute, Manual Series No. 4, 1962.
38. Day, H. L., "A Progress Report on Studies of Degrading Basalt Aggregate Bases," Highway Research Board Bulletin, 344:8-16, 1962.
39. Lambe, T. William, Soil Testing for Engineers, John Wiley & Sons, Inc., New York, pp. 29-42, 1951.
40. Gordon, J. L., "Anti-Stripping Agents - Pro and Con," Roads and Streets, December, 1955, pp. 121-123.
41. _____, "Method of Test for Maximum Specific Gravity of Bituminous Paving Mixtures," The Asphalt Institute Manual Series No. 2, Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types. Second Edition, 1962. Third Printing, 1963, Appendix C, pp. 162-165.
42. Hveem, F. N., "Use of the Centrifuge Kerosene Equivalent as Applied to Determine the Required Oil Content for Dense Graded Bituminous Mixtures," Proceedings of the Association of Asphalt Paving Technologists, 13:9-40, 1942.
43. Erickson, L. F., "Designing Durable Pavements," State of Idaho Department of Highways. An unpublished paper presented at the Seventh Annual Asphalt Conference, University of Idaho, Moscow, Idaho, October, 1966.
44. Monismith, Carl L., Kenneth E. Secor, and Edward W. Blackmer, "Asphalt Mixture Behavior in Repeated Flexure," Proceedings of the Association of Asphalt Paving Technologists, 30:188-222, 1961.
45. Hveem, F. N., and R. M. Carmany, "The Factors Underlying the Rational Design of Pavements," Highway Research Board Proceedings, 28:101-136, 1948.
46. Steel, Robert G. D., and James H. Torrie, Principals and Procedures of Statistics, McGraw-Hill Book Co., New York, 1960.
47. Endersby, V. A., and B. A. Vallerga, "Laboratory Compaction Methods and their Effects on Mechanical Stability Tests for Asphaltic Pavements," Proceedings of the Association of Asphalt Paving Technologists, 21:298-348, 1952.

48. Pauls, J. T., and J. F. Goode, "Application and Present Status of the Immersion-Compression Test," Proceedings of the Association of Asphalt Paving Technologists, 16:373-393, 1947.

APPENDIX A
INVESTIGATION PROCEDURE

I. Preparation and Make-Up of Standard Mineral Aggregate from
Idaho Department of Highways Pit Source Ada 53

A. Gradation

1. Grade the entire sample in accordance with Idaho T-1-65 using the 1-in., 3/4-in., 5/8-in., 1/2-in., 3/8-in., and No. 4 sieves.
2. Place the No. 4, 3/8-in., 1/2-in., and 5/8-in. material in separate sacks by individual sieve size.
3. Split all the minus No. 4 material into two parts, A and B, using the sample splitter.
4. Split part A into two parts, C and D, and part B into two parts, E and F, making a total of four approximately equal parts of material.
5. Combine parts C and F and split the combined material into two equal parts. Also combine parts D and E and split the combined material into two equal parts.
6. Split each of the resulting four equal parts into two additional parts making a total of eight approximately equal parts, parts one through eight.
7. Determine the gradation of parts 1, 3, 5, and 7 of the minus No. 4 material in accordance with Idaho T-1-65 by the method of hand quartering using the No. 4, No. 6, No. 8, No. 20, No. 30, No. 40, No. 50, No. 100, and No. 200 sieves.

8. Place the eight parts of minus No. 4 material into sacks and keep track of the sacks by part number. 101

B. Make-Up

1. If the standard aggregate is of uniform gradation meeting the Idaho Department of Highways 1965 Standard Specifications for both a Class "D" Plant-Mix Surface Course and a 3/4-in. Maximum Type "B" Aggregate Base Course, use the original uniform gradation to make up the specimens for all tests. Otherwise, combine the different sizes of plus No. 4 aggregate with the minus No. 4 aggregate to obtain a uniform gradation meeting the Class "D" Plant-Mix Surface Course specifications, and if possible, also the 3/4-in. Maximum Type "B" Aggregate Base Course specifications.
2. Make up the necessary samples from each of parts 1, 3, 5, and 7 of the minus No. 4 material and perform the following tests on each part:
 - a) Sand Equivalent Test - Idaho T-2-62
 - b) Liquid Limit Test - AASHO T-89-60
 - c) Plastic Limit Test - AASHO T-90-56
 - d) Plasticity Index Test - AASHO T-91-54
 - e) Fine Specific Gravity Test - Idaho T-75-62
 - f) Average Specific Gravity Test - IDH Form DH-897
 - g) Idaho Degradation Test - Idaho T-15-60
3. Make up the necessary samples from the coarse aggregate using the gradation determined in step 1 to perform each one of the following tests:
 - a) Coarse Aggregate Adsorption Test - Idaho T-76-62
 - b) Coarse Specific Gravity Test - Idaho T-76-62
 - c) Los Angeles Abrasion Test - AASHO T-96-60

4. Make up the necessary sample from the aggregate using the gradation determined in step 1 to perform the following test:
 - a) Bulk-Impregnated Specific Gravity Test. Procedure to be followed is contained in the report, "Specific Gravity of Aggregates in Asphaltic-Paving Mixtures," Highway Research Board Proceedings, 34:320-328, 1955.

II. Determination of Optimum Asphalt Ratio from Trial Mix Specimens

A. Procedure

1. Determine the Centrifuge Kerosene Equivalent values for each of the different asphalt mixture combinations in accordance with Idaho T-36-63 using the standard aggregate, 85-100 penetration asphalt cement, and the following types and amounts of mineral filler by weight of aggregate:
 - a) No filler
 - b) 1%, 2.5%, and 4% hydrated lime
 - c) 2%, 4%, and 6% portland cement
 - d) 2%, 5%, and 8% limestone dust
2. Examine the Centrifuge Kerosene Equivalent values for the different asphalt-mineral filler-aggregate combinations to determine the asphalt ratios at which each different trial mixture series should be made.
3. Select four appropriate asphalt ratios as whole percentages greater than and less than the Centrifuge Kerosene Equivalent value and make a trial mixture series in accordance with Idaho T-9-58 and Idaho

T-25-64 for each different asphalt-mineral filler-aggregate combination and mineral filler percentage shown in step 1.

4. Determine the following weights and measurements for each specimen in each trial mixture series:
 - a) Measure the height, H , of specimen in the mold to the nearest 0.001 in. after the leveling load has been applied.
 - b) Measure the mold diameter, D , to the nearest 0.001 in. when heated to 230° F.
 - c) Record the weight of aggregate, W_{agg} , the weight of mineral filler, W_{mf} , and the weight of asphalt, W_a , used to make the specimen.
5. Determine the unit weight of aggregate plus mineral filler, W_u , in each specimen of each trial mixture series in pounds per cubic foot as follows:

$$W_u = \frac{\text{Wt. of aggregate} + \text{Wt. of mineral filler}}{\text{Volume of Specimen}}$$

$$= \frac{(W_{agg} + W_{mf})}{(\pi \frac{D^2}{4} H)} \div \frac{453.6}{1728} \left(\frac{\text{lb.}}{\text{ft.}^3} \right)$$

6. Weigh the specimen to the nearest 0.1 gram, W_s , after it has been removed from the stabilometer.
7. Calculate the weight per unit volume of the specimen, W_o , in pounds per cubic foot as follows:

$$W_o = \frac{\text{Weight of Specimen}}{\text{Volume of Specimen}} = \frac{W_s}{(\pi \frac{D^2}{4} H) \div 1728} \div 453.6 \left(\frac{\text{lb.}}{\text{ft.}^3} \right)$$

8. Determine the specific gravity of the asphalt mixture, G_m , in each specimen of each trial mixture series by the maximum specific gravity method (Rice Method) as contained in Appendix C of the Asphalt Institute Publication, "Mix Design Methods for Asphalt Concrete and other Hot-Mix Types," (Manual Series No. 2, Second Edition, February, 1962, Third Printing, May, 1963) pp. 162-165.

9. Determine the air voids in each specimen of each trial mixture series as follows:

$$\text{Air voids} = \left[100 - \frac{W_o}{(G_m \times 62.4)} \times 100 \right] \%$$

10. Plot the relative stability, unit weight of aggregate plus mineral filler, and air voids for each different trial mixture series, and select the optimum asphalt ratio for each case.

III. Selection of Asphalt Ratio for Test Specimens

A. Control Specimens

1. Prepare the test specimens containing no mineral filler at the optimum asphalt ratio by weight of aggregate selected in II-A-10 for that particular case.

B. Specimens Containing Mineral Filler

1. Prepare an initial set of specimens for each different test at the optimum asphalt ratio by weight of aggregate plus mineral filler selected for use with each

of the following types and amounts of mineral filler by weight of aggregate:

- a) Hydrated lime - 2.5%
- b) Portland cement - 4%
- c) Limestone dust - 5%

2. Determine the total solid volume of mineral filler and asphalt used in each specimen (of the initial set of specimens for each mineral filler and each different test) as follows:

$$V_t = V_f + V_a = \frac{W_f}{G_f \gamma} + \frac{W_a}{G_a \gamma}$$

where: V_t = total solid volume of asphalt and mineral filler in each specimen of each initial set of specimens.

V_f = solid volume of mineral filler in each specimen.

V_a = solid volume of asphalt in each specimen.

W_f = weight of mineral filler in each specimen.

W_a = weight of asphalt in each specimen.

γ = unit weight of water taken as 1 gram per cubic centimeter.

G_f = specific gravity of mineral filler used.

G_a = specific gravity of asphalt.

3. Determine the V_t for the specimens in each initial set of specimens for each different mineral filler for each different test.

4. Use the same V_t determined in step 3 for each specimen in the second and third sets of specimens for each different mineral filler for each different test as prepared in steps 5 and 6.
5. Prepare the second set of specimens for each different test using the weight of asphalt per specimen calculated below for each of the following types and amounts of mineral filler by weight of aggregate:
- a) Hydrated lime - 1%
 - b) Portland cement - 2%
 - c) Limestone dust - 2%

The weight of asphalt per specimen shall be:

$$W_a = \left(V_t - \frac{W_f}{\gamma G_f} \right) \gamma G_a$$

using the appropriate V_t determined in step 3.

6. Prepare the third set of specimens for each different test using the weight of asphalt per specimen calculated below for each of the following types and amounts of mineral filler by weight of aggregate:
- a) Hydrated lime - 4%
 - b) Portland cement - 6%
 - c) Limestone dust - 8%

The weight of asphalt per specimen shall be:

$$W_a = \left(V_t - \frac{W_f}{\gamma G_f} \right) \gamma G_a$$

using the appropriate V_t determined in step 3.

7. Steps 1 through 6 are illustrated for a hypothetical case as follows:

Test to be performed: Relative Stability Test

Weight of aggregate in specimen = $W_g = 1156 \text{ gm.}$

Amount and type of mineral filler to be used:

4% portland cement

Optimum asphalt ratio:

5.9%

Specific gravity of mineral filler:

3.10

Specific gravity of asphalt:

1.00

Step 2:

$$W_f = 0.04 \times 1156 \text{ gm.} = 46.2 \text{ gm.}$$

$$W_a = 0.059 \times (1156 + 46.2) = 70.9 \text{ gm.}$$

$$V_t = \frac{46.2 \text{ gm.}}{1 \frac{\text{gm}}{\text{cc}} \times 3.1} + \frac{70.9 \text{ gm.}}{1 \frac{\text{gm}}{\text{cc}} \times 1.0} = 14.9 \text{ cc} + 70.9 \text{ cc} = 85.8 \text{ cc}$$

Step 5:

$$W_f = 0.02 \times 1156 \text{ gm.} = 23.1 \text{ gm.}$$

$$W_a = \left(85.8 \text{ cc} - \frac{23.1 \text{ gm.}}{1 \frac{\text{gm}}{\text{cc}} \times 3.1} \right) 1 \frac{\text{gm}}{\text{cc}} \times 1.0 = 85.8 \text{ gm.} - 7.5 \text{ gm.} = 78.3 \text{ gm.}$$

Step 6:

$$W_f = 0.06 \times 1156 \text{ gm.} = 69.4 \text{ gm.}$$

$$W_a = \left(85.8 \text{ cc} - \frac{69.4 \text{ gm.}}{1 \frac{\text{gm}}{\text{cc}} \times 3.1} \right) 1 \frac{\text{gm}}{\text{cc}} \times 1.0 = 85.8 \text{ gm.} - 22.4 \text{ gm.} = 63.4 \text{ gm.}$$

The three asphalt ratios involved would then be:

a) For 2% portland cement:

$$\frac{78.3 \text{ gm.}}{(1156 + 23.1)} \times 100 = 6.6\%$$

b) For 4% portland cement:

$$\frac{70.9 \text{ gm.}}{(1156 + 46.2)} \times 100 = 5.9\%$$

c) For 6% portland cement:

$$\frac{63.4 \text{ gm.}}{(1156 + 69.4)} \times 100 = 5.2\%$$

IV. Relative Stability Test

A. Number of Test Specimens Required

1. One set of five control test specimens containing no mineral filler.
2. One set of five test specimens for each type and amount of mineral filler indicated in III-B.

B. Preparation of Test Specimens

1. Prepare each set of test specimens in accordance with Idaho T-9-58 and Idaho T-25-64 with the following modifications:
 - a) Select from the trial mixture series made in Section II the weight of aggregate, W_g , that will make a specimen containing a mineral filler 2.5 ± 0.1 in. high and use this $W_g \pm 1$ gm. for all test specimens prepared for this test.

- (1) Record the actual weight of aggregate used.
- b) Select the optimum asphalt ratio determined from the trial mixture series containing no filler in II-A-10 as the asphalt ratio for the set of control test specimens.
- c) Using the weight of aggregate selected in part a), determine the asphalt ratio for each set of specimens containing a mineral filler in accordance with the procedure indicated in III-B.
- d) Place the mineral filler with the aggregate, heat the combined aggregate and mineral filler mixture to $275^{\circ}\text{F.} - 300^{\circ}\text{F.}$, and mix the heated combination for 30 seconds before adding the asphalt.
- e) An initial specimen shall be made for the purpose of "buttering" each mold. The specimen shall be discarded and the inside of the mold shall be cleaned of mixture residue by wiping with a clean dry cloth.
- f) Make the first specimen in the set of test specimens containing $\frac{1}{4}$ per cent portland cement by weight of aggregate and determine the volume, V_s , of the specimen in cubic inches.
- g) Prepare the remainder of the test specimens required in steps 1 and 2 of Sub-Section A at the same constant volume, V_s , determined in part f).
- h) The constant volume, V_s , is to be attained for each specimen in the following manner:
 - (1) Determine the diameter, D_1 , of each mold to the nearest 0.001 in. when heated to 230°F.
 - (2) Calculate the necessary height, H_1 , of each specimen to the nearest 0.001 in. as follows:

$$H_1 = \frac{1.4 V_s}{\pi D_1^2} = 1.2732 \frac{V_s}{D_1^2} \quad (\text{inches})$$

- (3) Apply the leveling load after compaction of the specimen required to attain the height, H_1 , and record the amount of the leveling load.
- (4) Use an Ames dial to determine when the height, H_1 , has been attained.
 - (a) Determine the height measurement at four different locations on the top surface of the specimen.
 - (b) Use the average of the four height measurements as the height, H_1 .
2. Record all data necessary in the preparation of the specimens including all calculations required in Sections II and III.

C. Performance of Test

1. Perform the Relative Stability Test on each specimen of each set in accordance with Idaho T-9-58 and record all data.
2. After the first and fifth specimens of each set have been removed from the stabilometer, determine the specific gravity of the asphalt mixture, G_m , by the maximum specific gravity method indicated in II-A-8.
3. Determine the amount of moisture and volatiles in the second, third, and fourth specimens of each set in the following manner:

- a) As the second and third specimens of the set are removed from the stabilometer, cover them to prevent further loss of moisture and volatiles, and place them in a tared pan.
- b) After the fourth specimen has been removed from the stabilometer, place it in the pan, uncover the other two specimens, and weigh the tared pan with the three specimens. Record the weight as W_1 .
- c) Subtract the weight of the pan to obtain the initial weight of the three specimens, W_2 .
- d) Place the pan with the specimens in a 230° F. oven for 1 hour.
- e) Break up the three specimens after they have been in the oven for 1 hour to facilitate the drying process.
- f) Return the pan with the specimens to the 230° F. oven for a 23-hour drying period.
- g) Remove the pan with the specimens from the oven, weigh, and record the weight as W_3 .
- h) Subtract the weight of the pan to obtain the final weight of the specimens, W_4 .
- i) Compute the average percentage of moisture and volatiles (M & V) in the specimens as follows:

$$\begin{aligned}
 \% \text{ M \& V (Specimen)} &= \frac{\text{Loss in Weight}}{\text{Dry Weight of Specimen}} \times 100 \\
 &= \frac{W_2 - W_4}{W_4} \times 100 \quad (\%)
 \end{aligned}$$

V. Moisture Vapor Susceptibility Test

A. Number of Test Specimens Required

1. One set of five control test specimens containing no mineral filler.
2. One set of five test specimens for each type and amount of mineral filler indicated in III-B.

B. Preparation of Test Specimens

1. Mix, fabricate and compact each set of test specimens in stainless steel molds in accordance with the procedure outlined in IV-B with the exception that all specimens shall be prepared with the volume, V_s , determined in part f) of step 1.
2. Prepare each compacted specimen for testing in accordance with Section C, Preparation of Sample, of Test Method No. California 307-C, "Method of Test for Moisture Vapor Susceptibility of Bituminous Mixtures," State of California Department of Public Works, Division of Highways, Materials Manual, Testing and Control Procedures, Volume I.

C. Performance of Test

1. Place the assembly in a 140° F. oven for a continuous period of 75 hours. Water is to be continuously maintained in the pan during the 75-hour period.

2. Remove the assembly from the oven and immediately obtain the relative stability value in accordance with Idaho T-9-58. Record all data.
3. After the first and fifth specimens of each set have been removed from the stabilometer, determine the specific gravity of the asphalt mixture, G_m , by the maximum specific gravity method indicated in II-A-8.
 - a) After determining the weight in grams of the sample in water, dry the specimen in the 230° F. oven for 24 hours before determining the weight in grams of the dry specimen in air.
4. Determine the amount of moisture and volatiles in the second, third, and fourth specimens of each set in the manner indicated in IV-C-3.
5. Calculate the average percentage of moisture, M, gained by the specimens during this test as follows:

$$M = \left(\% M \& V (\text{Specimen})_{MVS} - \% M \& V (\text{Specimen})_{RS} \right) \text{ in } \%$$

where: $\% M \& V (\text{Specimen})_{MVS}$ = average percentage of moisture and volatiles in the specimens from the Moisture Vapor Susceptibility Test.

$\% M \& V (\text{Specimen})_{RS}$ = average percentage of moisture and volatiles in the specimens from the Relative Stability Test.

VI. Minnesota Cold Water Abrasion Test

A. Number of Test Specimens Required

1. One set of eight control test specimens containing no mineral filler.
2. One set of eight test specimens for each type and amount of mineral filler indicated in III-B.

B. Preparation of Test Specimens

1. The size of the individual batches of mixture shall be the amount required for one set of test specimens.
2. Compute the weight of mixture in grams for the specimens of the sets required in steps 1 and 2 of Sub-Section A as follows:

$$\begin{aligned}
 W_{cw} &= 0.9 \frac{V_{cw}}{V_s} A = 0.9 \frac{\pi \frac{d^2}{4} h}{V_s} A = 0.9 \frac{\pi \frac{d^2}{4}}{V_s} A \\
 &= 1.414 \frac{A d^2}{V_s} \text{ gm.}
 \end{aligned}$$

where: W_{cw} = weight in grams of the specimen.
 V_{cw} = volume in cubic inches of the specimen.
 d = diameter of standard 2-in. Hubbard-Field Mold to the nearest 0.001 in. when heated to 260° F.
 h = height of specimen = 2.000 in.
 A = weight in grams of dry mixture in air (of the first specimen in the corresponding set of specimens with the

same type and amount of mineral filler prepared for the Relative Stability Test) as determined from the maximum specific gravity method computed in IV-C-2.

V_s = volume of the relative stability specimen in cubic inches as determined in IV-B-1-f.

3. Calculate the weight of aggregate, W_r , in the specimen of each set as follows:

$$W_r = \frac{W_{cw} (1 + P)}{\left(1 + \frac{\% f}{100} + \frac{\% a}{100}\right)}$$

where: % f = percentage of mineral filler by weight of aggregate in specimen.

% a = percentage of asphalt by weight of aggregate in specimen.

P = percentage of aggregate retained on the 1/2-in. screen.

4. Use the weight of aggregate, W_r , determined for the set with 2.5 per cent hydrated lime, the W_r for the set with 4 per cent portland cement, and the W_r for the set with 5 per cent limestone dust to prepare the specimens in the three sets containing different amounts of hydrated lime, portland cement, and limestone dust, respectively.
5. Mix each individual batch in accordance with Idaho T-25-64 keeping the heating and mixing temperature at $265^\circ \text{F.} \pm 5^\circ \text{F.}$
 - a) Place the mineral filler with the aggregate, heat the combined aggregate and mineral filler to

275° F. - 300° F., and mix the heated combination for 30 seconds before adding the asphalt.

6. Maintain the temperature of the asphalt mixture at 265° F. \pm 5° F. during the sieving and compaction procedures.
7. Sieve the asphalt mixture on the heated 1/2-in. sieve and use only that part of the mixture passing the 1/2-in. sieve to mold the specimens.
8. Place the 1/2-in. minus portion of the mixture in a 2-in. diameter Hubbard-Field Mold, rod thoroughly with a 1/4-in. diameter rod, and use a properly designed metal follower to compress the mixture into a specimen 2.000 in. in height, (h).
9. Use an Ames dial to determine when the 2.000 in. height has been attained and maintain the compression load for 1 minute at the load necessary to keep the height, h, at 2.000 in.
10. Using a fan, cool the compacted specimen in air to room temperature before removing the specimen from the mold.
11. Remove the specimen from the mold and weigh it to the nearest 0.1 gram.
12. Place the eight specimens comprising the set together in a pan and place the pan with the specimens in a 140° F. oven for 24 hours.

C. Performance of Test

1. Remove the eight specimens from the oven and immerse them in a 120° F. water bath for 6 days.
2. Remove the specimens from the water bath, cool to room temperature to prevent damage when handling, surface-dry, and weigh each specimen to the nearest 0.1 gram.
3. Immerse the eight specimens in a water bath at 33° F. $\pm 1^\circ$ F. for 1 hour.
4. Place the set of eight specimens in a Deval cylinder previously cooled to 33° F. $\pm 1^\circ$ F. with ice water and fill the cylinder with water at 33° F. $\pm 1^\circ$ F. to a level of approximately 1 in. below the top of the cylinder.
5. Seal the cylinder and rotate the sealed cylinder for 1000 revolutions in a Deval Abrasion Machine.
6. Remove the specimens from the Deval cylinder, surface-dry, and weigh each specimen to the nearest 0.1 gram.
7. Compute the percentage weight loss due to abrasion as follows:

$$\text{Percentage abrasion loss} = \frac{B - C}{B} \times 100 \quad (\%)$$

where: B = original surface-dry weight of the eight specimens. (From step 2)

C = final surface-dry weight of the eight specimens. (From step 6)

VII. Immersion-Compression Test

A. Number of Test Specimens Required

1. Two sets of five control test specimens containing no mineral filler.
2. Two sets of five test specimens for each type and amount of mineral filler indicated in III-B.

B. Preparation of Test Specimens

1. The size of the individual batches of mixture shall be limited to the amount required for one test specimen.
2. Prepare a trial specimen for each of the different pairs of sets required in steps 1 and 2 of Sub-Section A.
3. Calculate the weight of aggregate, W_x , in the different trial specimens as follows:

$$W_x = \frac{4.0}{2.5} W_g = 1.6 W_g \text{ gm.}$$

where: W_g = weight of aggregate in grams selected for the relative stability specimen in IV-B-1-a.

4. Mold the trial specimen for each different pair of sets in accordance with ASTM Designation D-1074-60, and determine the actual height, h_g , of the specimen.
 - a) Mix the mineral filler with the dry aggregate at room temperature for 30 seconds, heat the mixture to $325^\circ \text{F.} \pm 5^\circ \text{F.}$, and add the asphalt.

5. If the height of each trial specimen representing the three pairs of sets containing different amounts of the same mineral filler is within the range of 4.0 ± 0.1 in., use the weight of aggregate, W_x , selected in step 3 to mold all of the specimens in the three pairs of sets for that type of mineral filler in accordance with ASTM Designation D-1074-60.
 - a) Record the mold diameter to the nearest 0.001 in. when heated to $255^\circ \text{F.} \pm 5^\circ \text{F.}$
 - b) Record the height of each specimen to the nearest 0.001 in.

6. If the heights of the three trial specimens for the same type of filler are not within the range 4.0 ± 0.1 in., adjust the weight of aggregate computed in step 3 to determine a new weight of aggregate in grams, W_z , such that the heights of the three new trial specimens are within the required range.
 - a) Use the weight of aggregate, W_z , to mold all of the specimens in the three pairs of sets for that type of mineral filler in accordance with ASTM Designation D-1074-60.
 - b) Record the mold diameter to the nearest 0.001 in. when heated to $255^\circ \text{F.} \pm 5^\circ \text{F.}$
 - c) Record the height of each specimen to the nearest 0.001 in.

C. Performance of Test

1. Perform the Immersion-Compression Test in accordance with ASTM Designation D-1075-54.
 - a) Use the group 2, Alternate Procedure under Section 5, Procedure, Sub-Section C of ASTM Designation

D-1075-54.

- b) Record all data and make all necessary calculations.

APPENDIX B

COMPLETE TESTS ON ASPHALT

Report of Tests on 85-100 Penetration Asphalt Cement

Submission No.	1	2	3	4
Quantity Represented in gallons	5	5	10	20

TEST RESULTS

Penetration of Orig. Sample at 77° F., 100 gm., 5 sec.	86	89	89	93
Flash Point, P.M.C.C. (° F.)	500+	500+	500	500+
Kinematic Viscosity at 275° F. (cs)	256	259.5	263	290
Specific Gravity at 77°/77° F.	1.014	1.018	1.012	1.022
Solubility in CCl ₄ (%)	99.69	99.66	99.84	99.81
Spot Test, Heptane Xylene Equivalent at 35% Xylene	Negative	Negative	Negative	Negative

TESTS ON RESIDUE FROM THIN FILM L.O.H.

Loss on Heating at 325° F., 5 hours (%)	0.0	0.0	0.0	0.0
Penetration at 77° F., 100 gm., 5 sec.	54	59	48	61
Ratio of Thin Film L.O.H. Pen./Orig. Pen. (%)	62.8	66.3	53.9	65.6
Ductility at 77° F., 5 cm/min (cm)	100+	140+	140+	150+

APPENDIX C

OPTIMUM ASPHALT RATIOS FOR TRIAL MIXTURE SPECIMENS

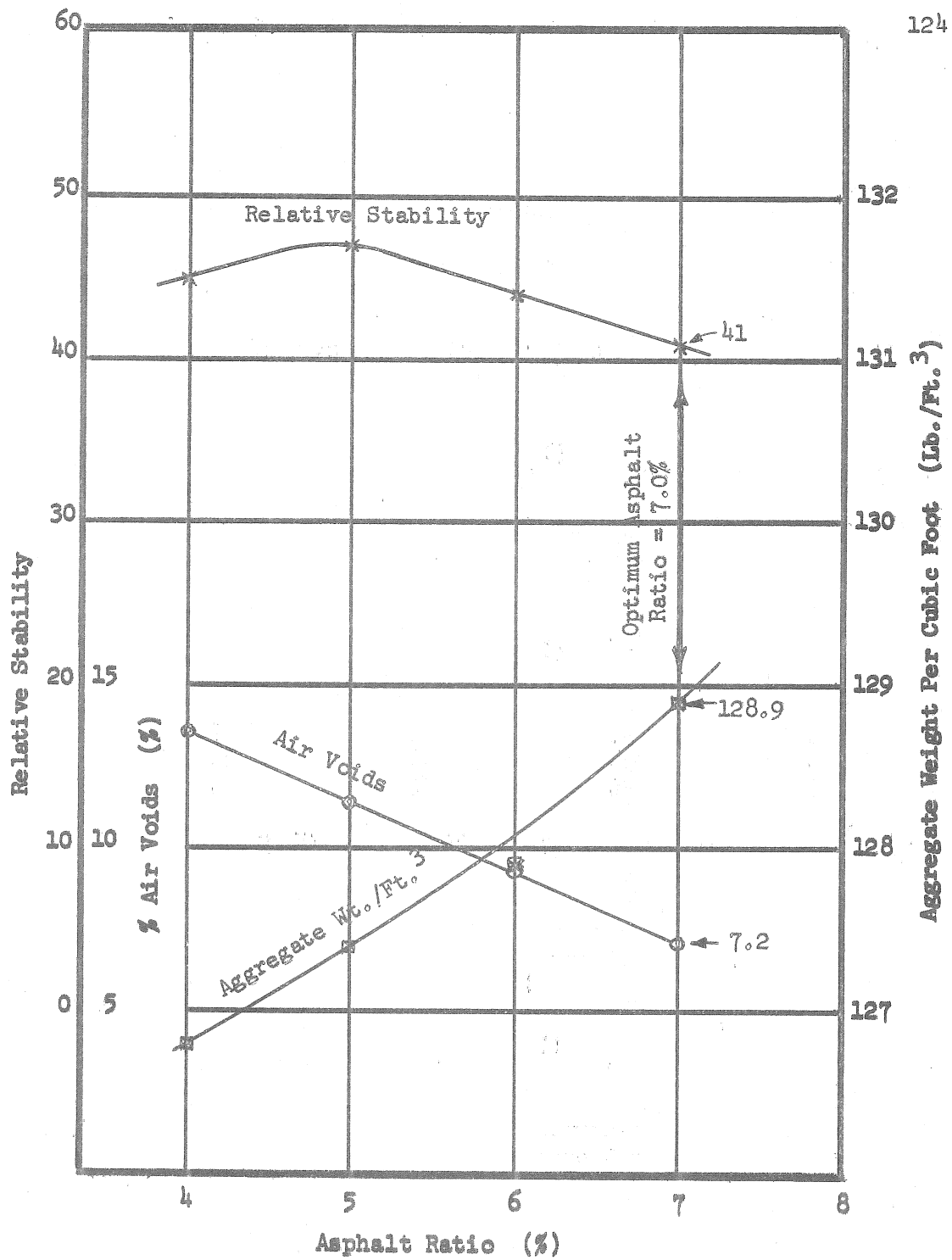


Figure 9. Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing No Mineral Filler

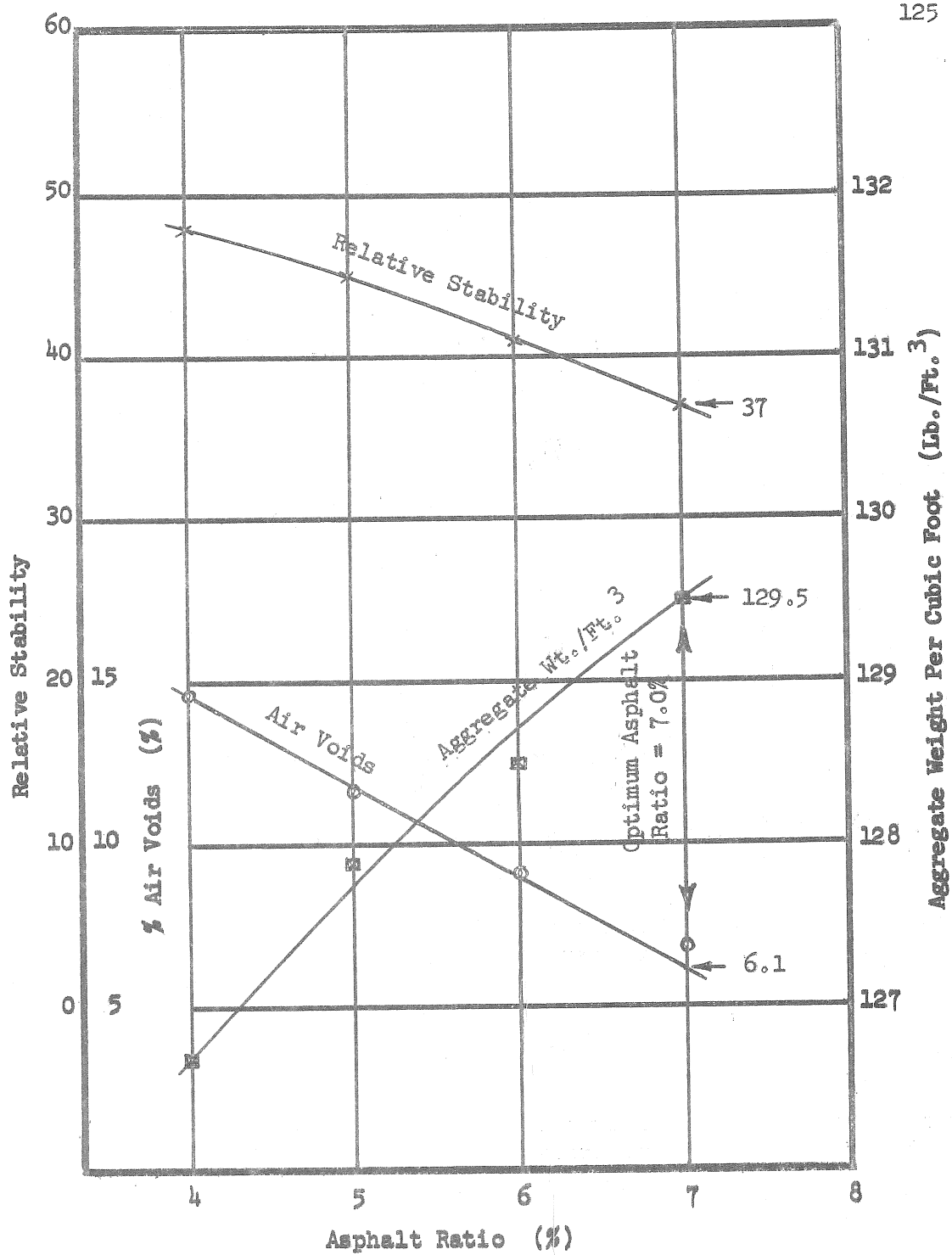


Figure 10. Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 1% Hydrated Lime

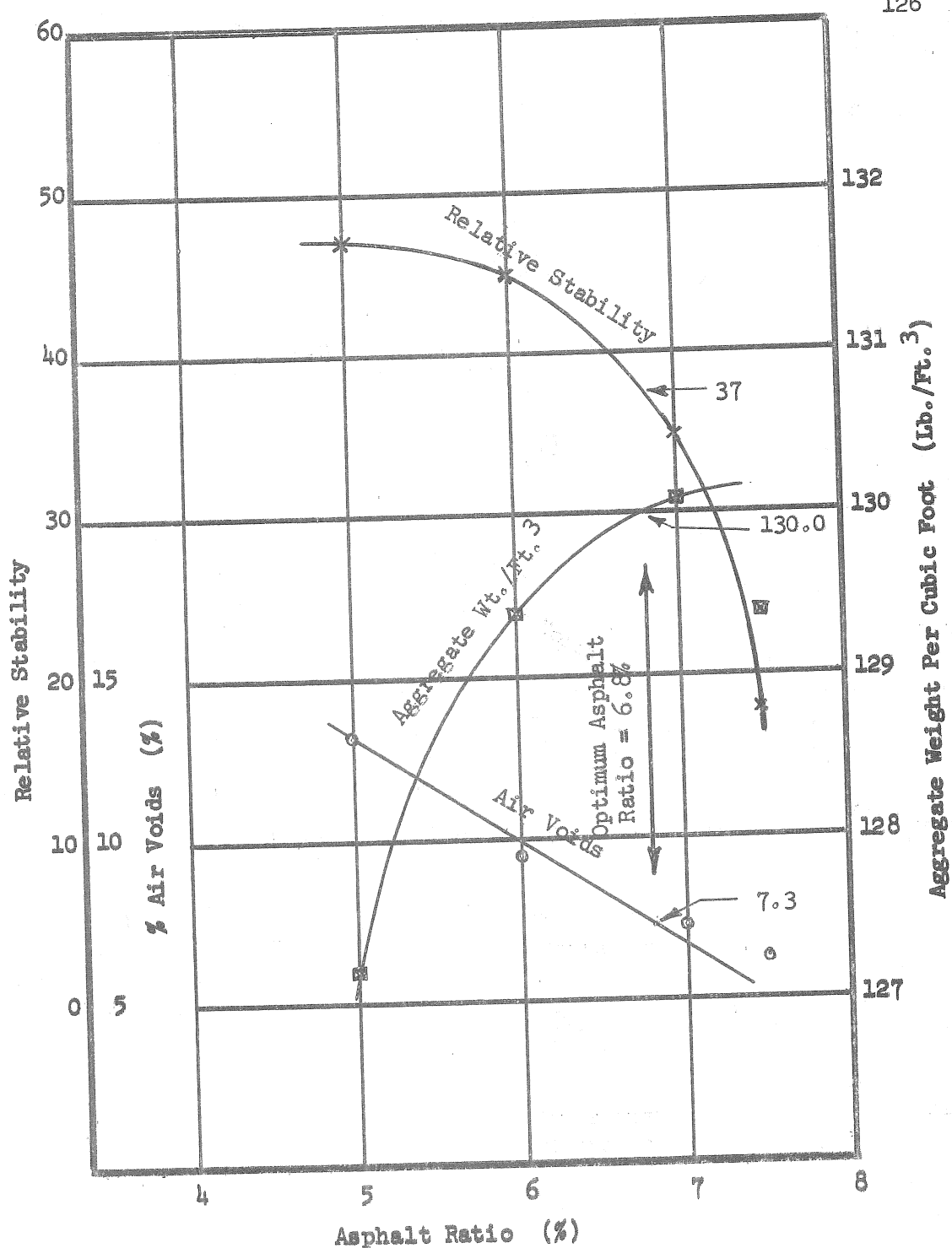


Figure 11. Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 2 1/2% Hydrated Lime

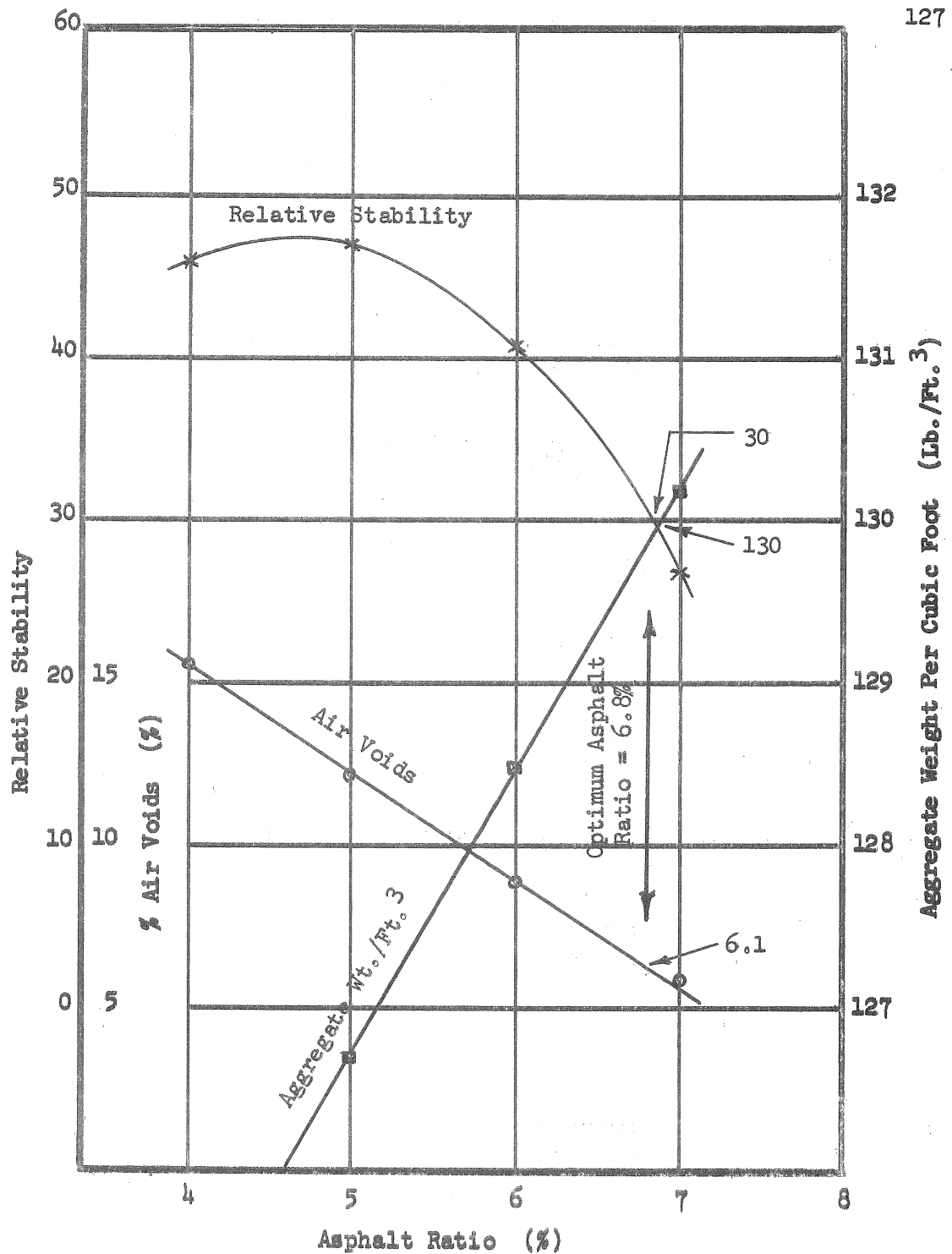


Figure 12. Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 3% Hydrated Lime

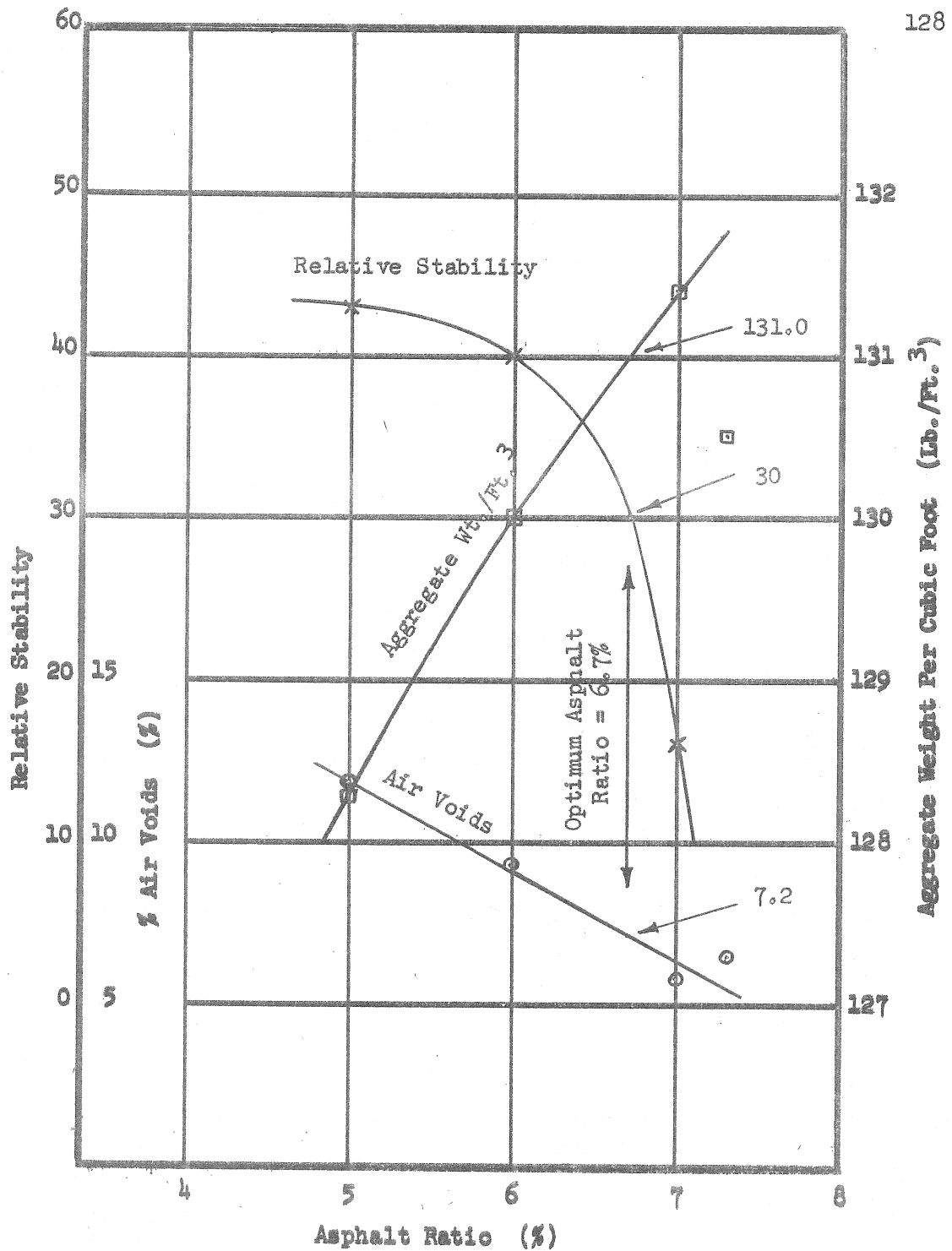


Figure 13. Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 4% Hydrated Lime

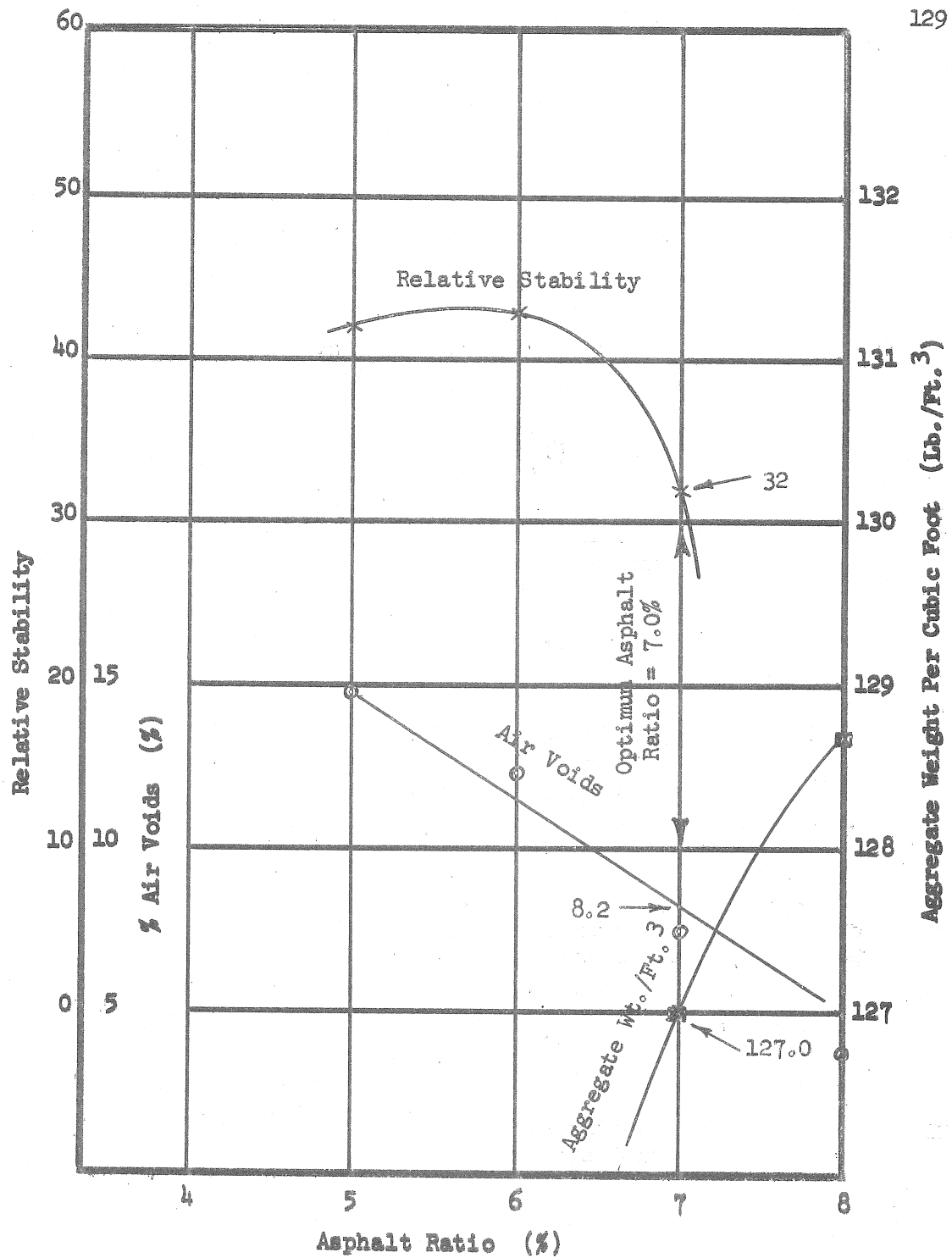


Figure 14. Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 6% Hydrated Lime

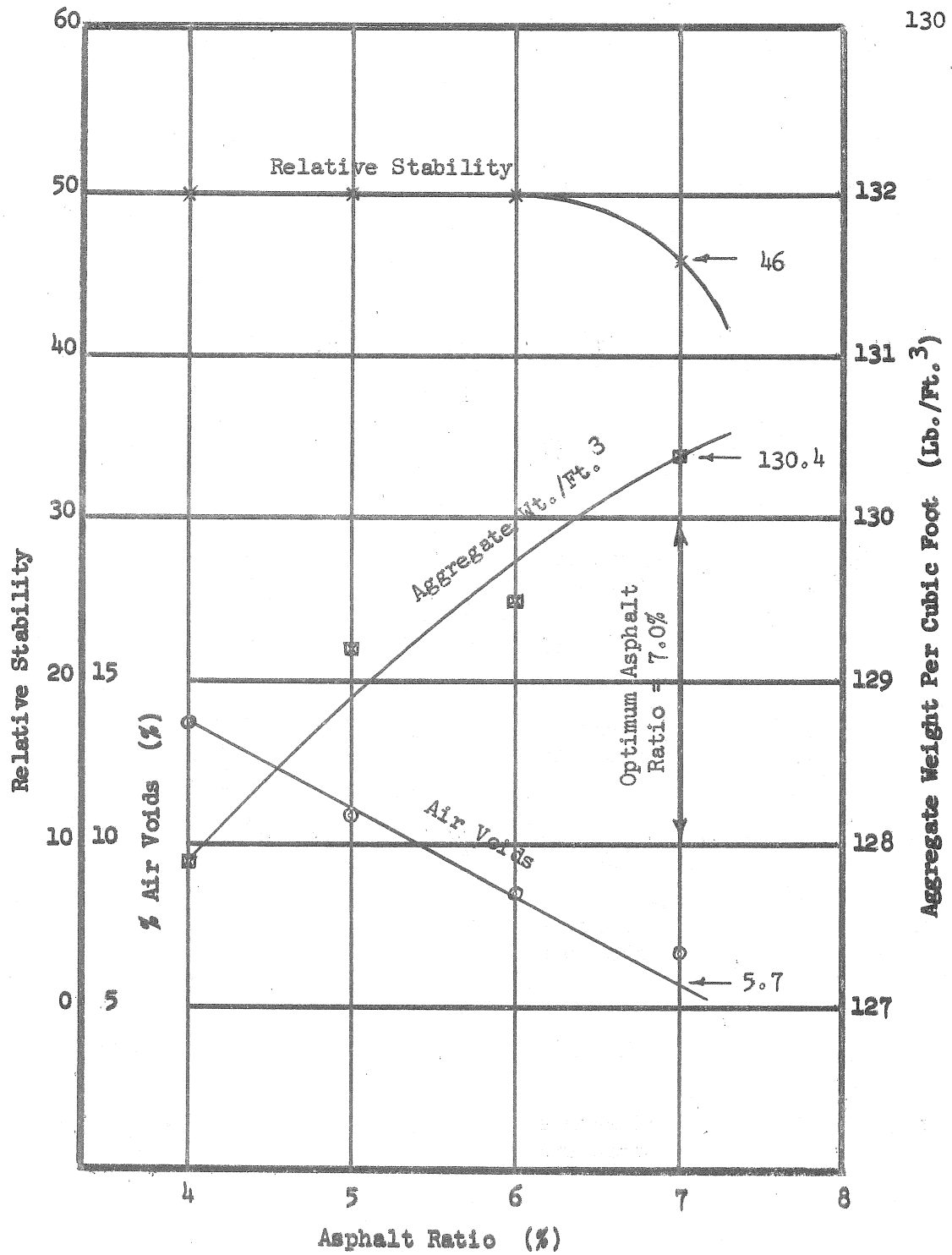


Figure 15. Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 2% Portland Cement

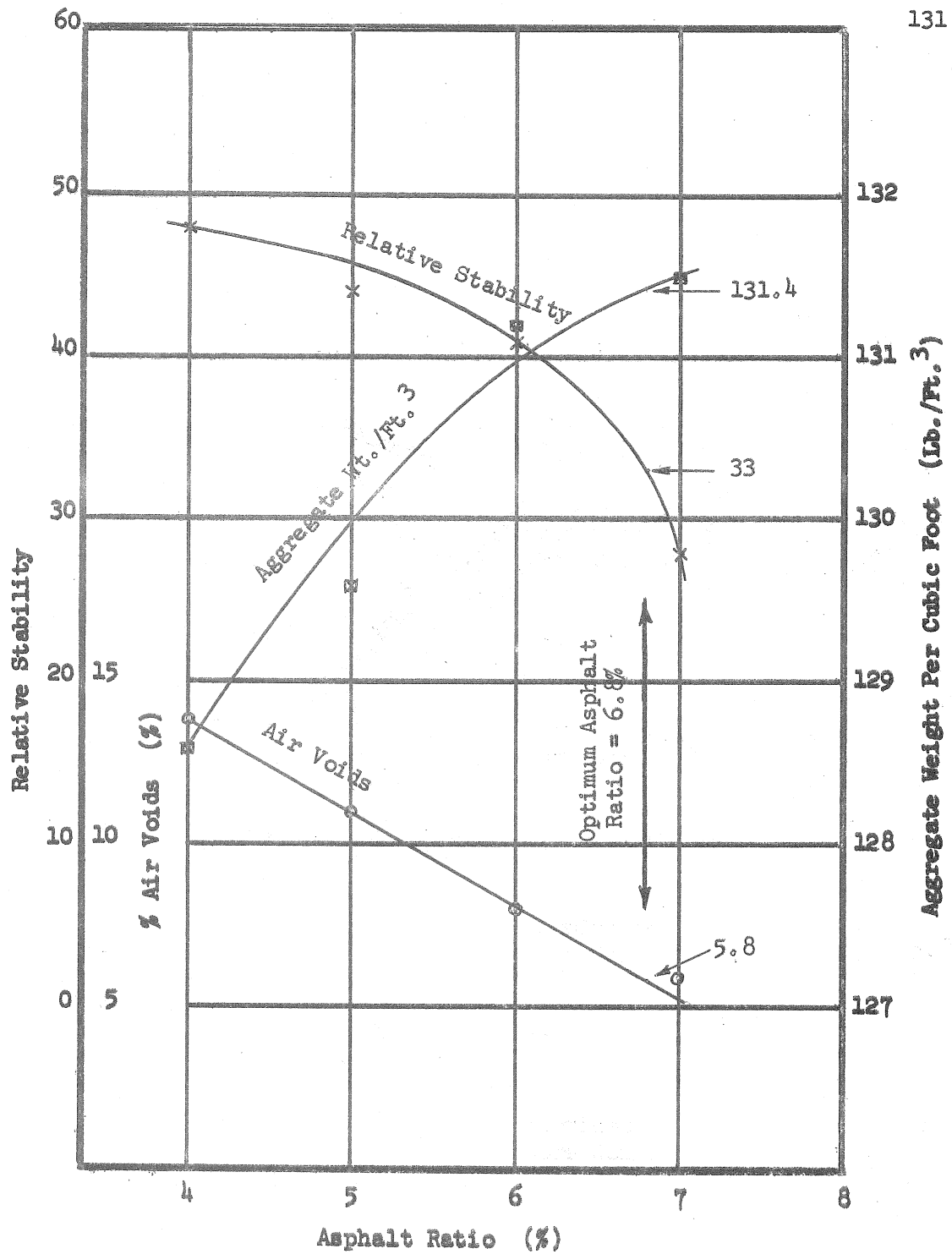


Figure 16. Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 4% Portland Cement

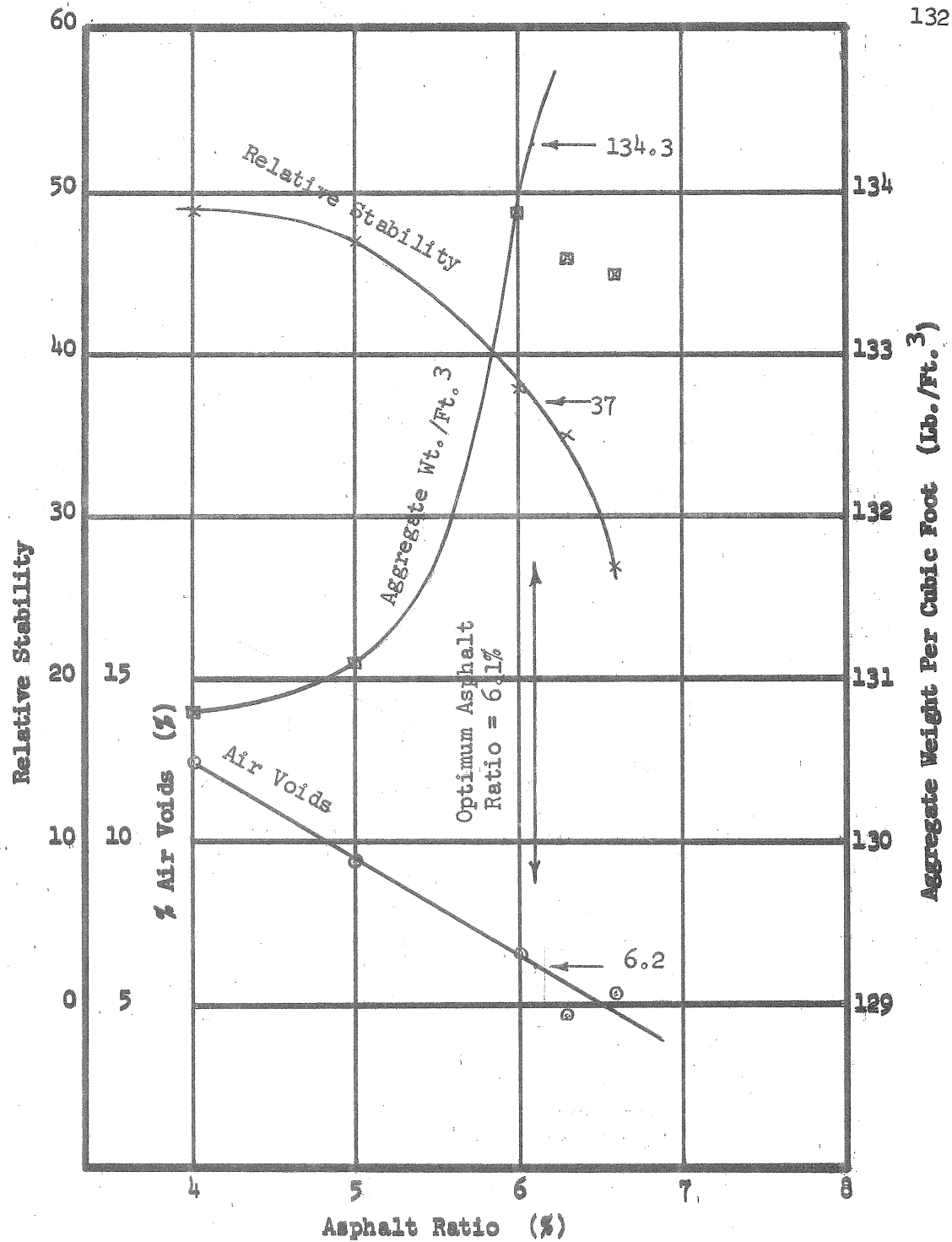


Figure 17. Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 6% Portland Cement

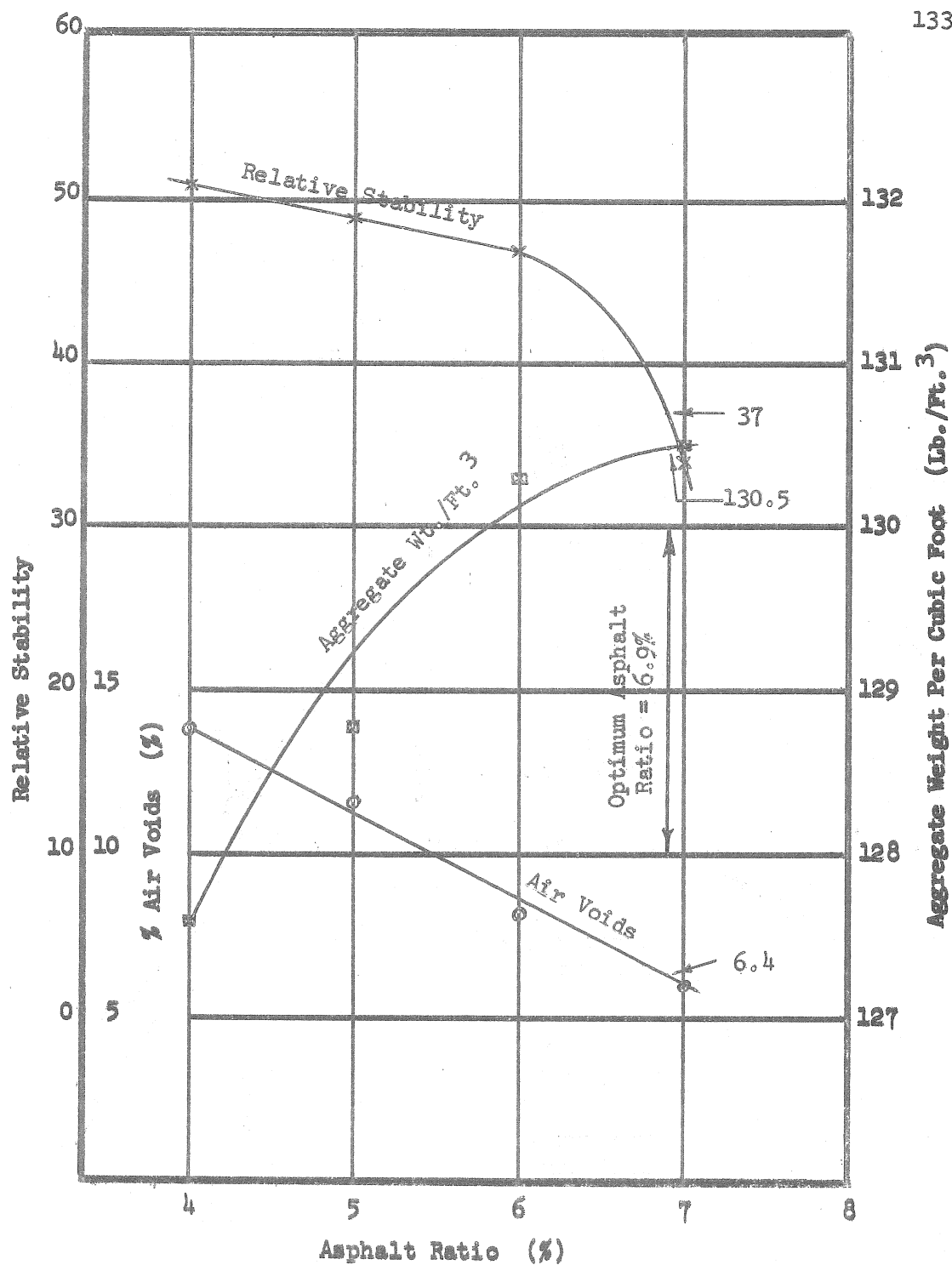


Figure 18. Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 2% Limestone Dust

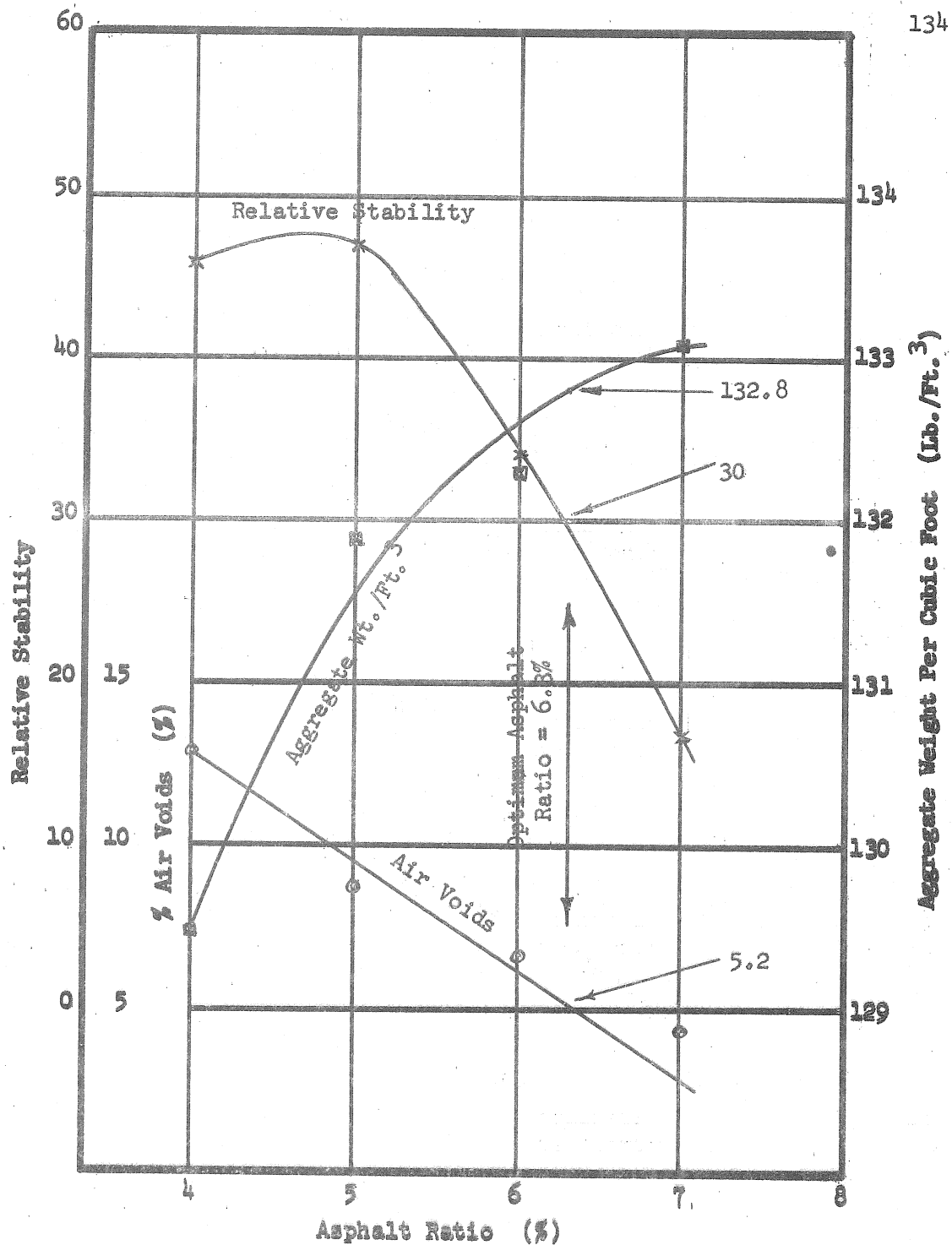


Figure 19. Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 5% Limestone Dust

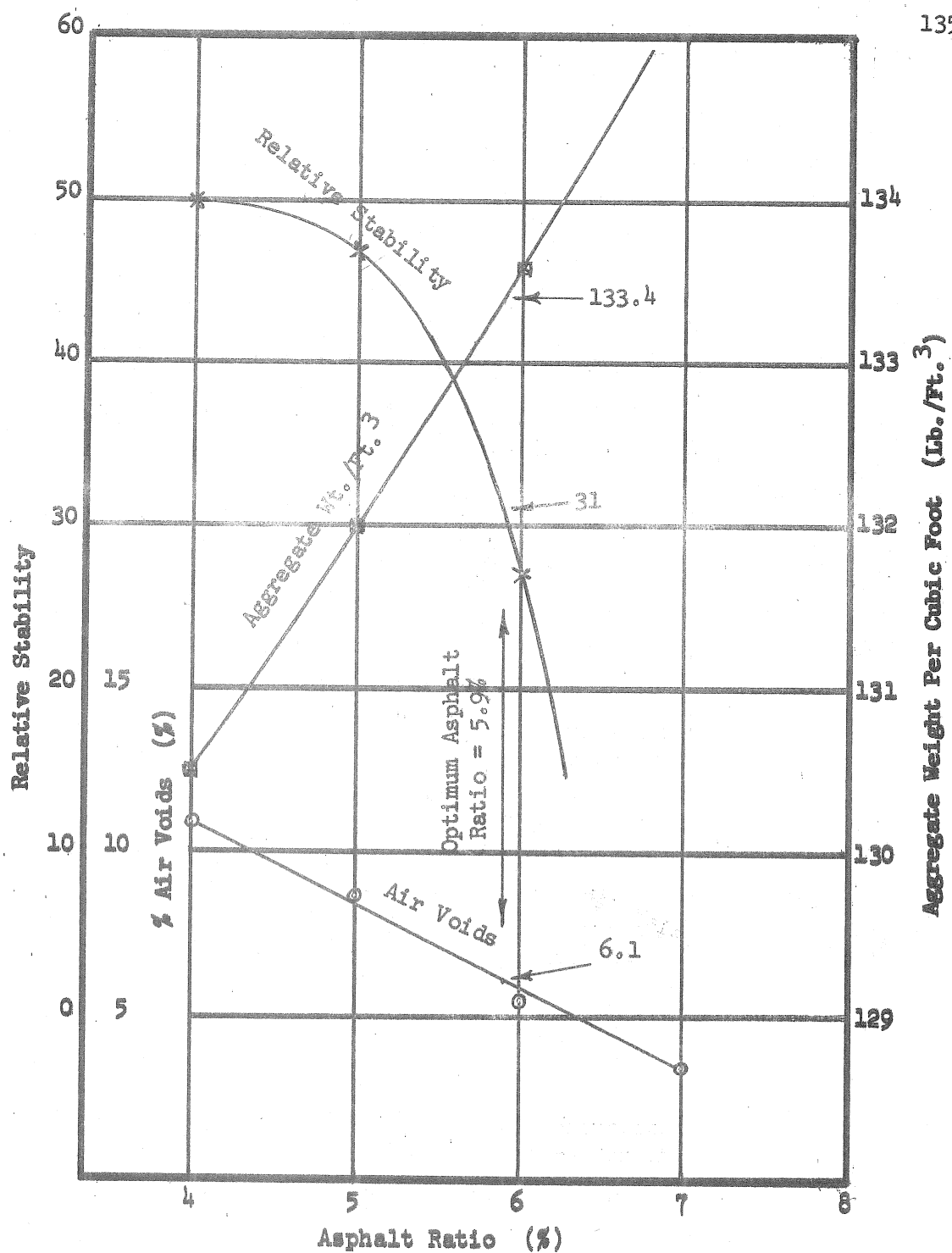


Figure 20. Trial Mixtures for Determination of Optimum Asphalt Ratio for Asphalt Mixture Containing 8% Limestone Dust

APPENDIX D
STATISTICAL ANALYSIS DATA

Duncan's New Multiple-Range Test for Relative Stability Test Mean Relative Stability Values
and Moisture Vapor Susceptibility Test Mean Relative Stability Values

PC MVS	2%	PC RS	2%	LD MVS	2%	LD RS	2%	PC MVS	4%	PC RS	5%	LD MVS	2%	LD RS	5%	PC MVS	5%	PC RS	2%	LD MVS	5%	LD RS	5%	PC MVS	2.5%	PC RS	1%	LD MVS	2.5%	LD RS	1%	PC MVS	4%	PC RS	4%	LD MVS	4%	LD RS	4%	PC MVS	1%	PC RS	6%	LD MVS	1%	LD RS	6%	PC MVS	6%	PC RS	6%	LD MVS	8%	LD RS	8%	PC MVS	8%	PC RS	8%	LD MVS	8%	LD RS	8%
--------	----	-------	----	--------	----	-------	----	--------	----	-------	----	--------	----	-------	----	--------	----	-------	----	--------	----	-------	----	--------	------	-------	----	--------	------	-------	----	--------	----	-------	----	--------	----	-------	----	--------	----	-------	----	--------	----	-------	----	--------	----	-------	----	--------	----	-------	----	--------	----	-------	----	--------	----	-------	----

Any mixtures not underscored by the
same line have significantly
different stability values

Legend

RS - Relative Stability Test

MVS - Moisture Vapor
Susceptibility Test

NF - No Filler

HL - Hydrated Lime

PC - Portland Cement

LD - Limestone Dust

LINEAR CORRELATION COEFFICIENTS

RS Test	MVS Test	RS Test	MVS Test	Immersion-Compression Test				MCWA Test	
				Planned		Indicated		Abrasion Loss	
RS	RS	M&V	M&V	DS	WS	IRS	DS	WS	IRS

RS Test-RS	MVS Test-RS	RS Test-M&V	MVS Test-M&V	Immersion-Compression Test			MCWA Abras. Loss	Plan. Ind.
				DS	WS	IRS		
1.00	0.79	0.15	0.62	0.30	0.28	0.03	0.28	0.56
1.00	0.04	0.47	0.47	0.42	0.35	-0.07	0.47	0.54
1.00	0.11	0.14	0.11	0.14	0.22	0.28	0.11	0.32
1.00	0.64	0.64	1.00	0.64	0.70	0.21	0.54	0.83
1.00	0.97	1.00	0.97	1.00	0.97	-0.06	0.91	0.84
1.00	1.00	1.00	1.00	1.00	1.00	0.19	0.84	0.86
1.00	1.00	1.00	1.00	1.00	1.00	-0.18	0.14	0.14
1.00	1.00	1.00	1.00	1.00	1.00	0.84	0.56	0.37
1.00	1.00	1.00	1.00	1.00	1.00	0.87	0.37	0.42
1.00	1.00	1.00	1.00	1.00	1.00	0.08	0.55	0.37
1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.37	1.00

Legend			
RS - Relative Stability	DS - Dry Strengths		
MVS - Moisture Vapor Susceptibility	WS - Immersed Strengths		
M & V- Moisture and Volatiles	IRS - Index of Retained Strength		
Planned - Planned Asphalt Ratio Specimens			
Indicated - Indicated Asphalt Ratio Specimens			
	MCWA - Minnesota Cold Water Abrasion		